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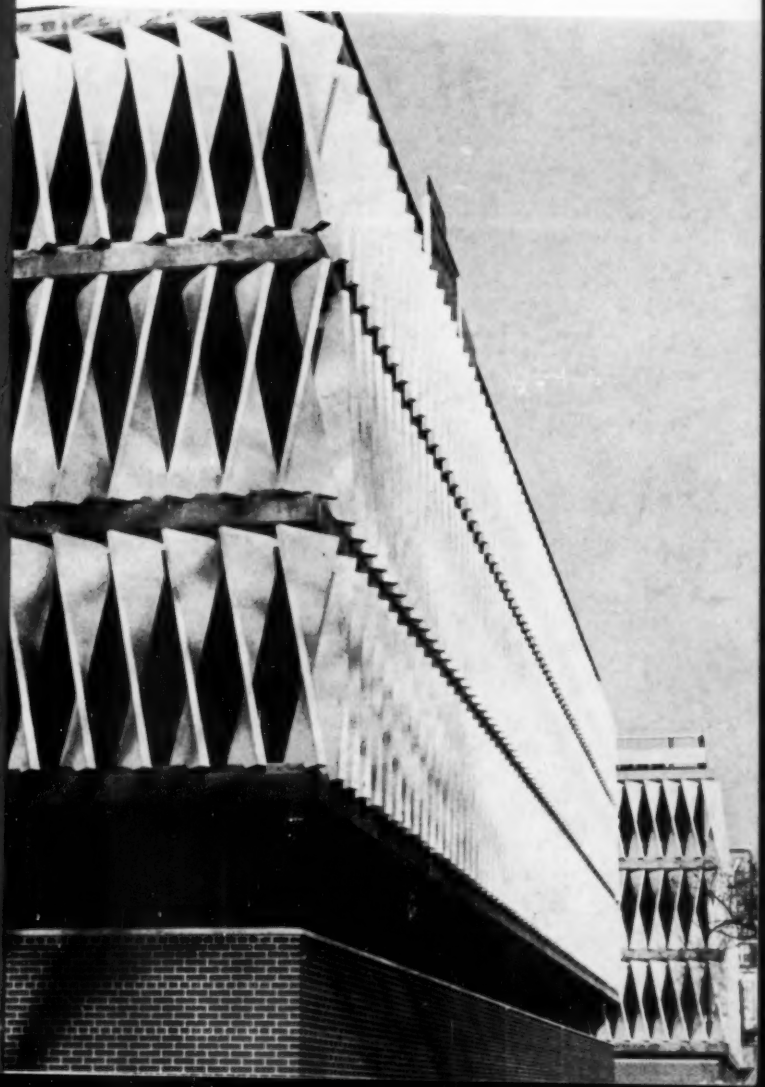

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Coauthors STANTON WALKER and DELMAR L. BLOEM report on their research on "Effects of Aggregate Size on Properties of Concrete." This paper was presented at the ACI 1960 annual convention.

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The late JOSEPH DI STASIO, SR., and M. P. VAN BUREN present a method for calculating the maximum unit shearing stresses in their paper, "Transfer of Bending Moment Between Flat Plate Floor Slab and Column." The values of allowable stresses are discussed, and recommendations made for a suggested test program.

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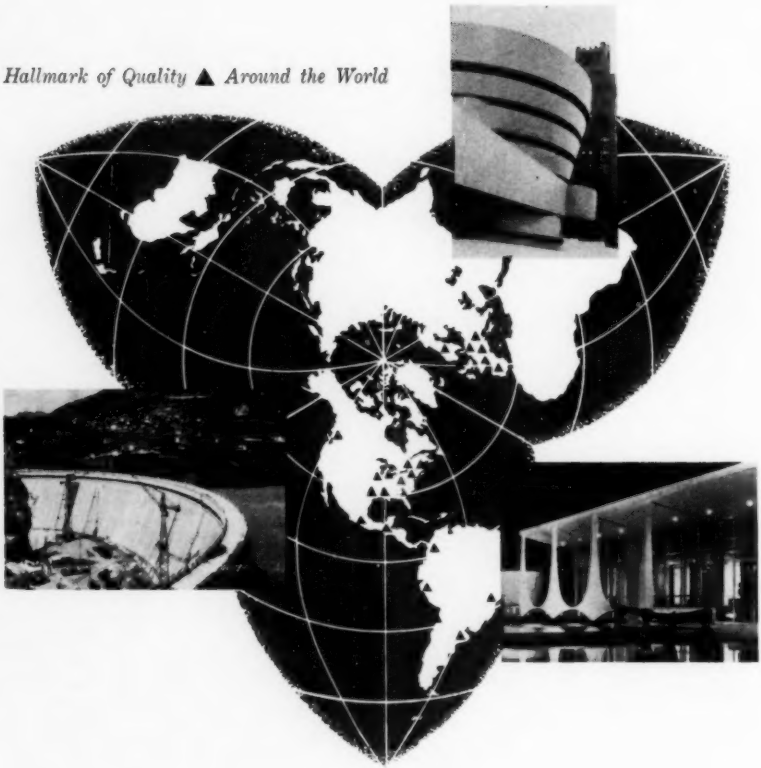

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Title No. 57-6

From ACI Committee 201 Symposium on
Restoration of Deteriorated Concrete

Conventional Methods of Repairing Concrete

By LEWIS H. TUTHILL

Some of the most important aspects of conventional methods of concrete repair and restoration are reviewed, compared, and emphasized. Particularly, factors affecting bond, permeability, cracking, durability, and appearance are discussed.

It is particularly emphasized that fully satisfactory repairs are seldom obtained without thorough inspection to insure strict adherence to all parts of the specified procedure.

■ NECESSITY FOR CONCRETE REPAIR arises either from imperfections in workmanship during construction or as a result of subsequent damage or deterioration. Probably for as long as we have been using concrete, conventional methods for making such repairs have been developing through trial and error and good and poor results. Conventional methods are those which employ the usual materials and tools of concrete construction, but with a certain refinement of procedure which experience has taught is necessary for good results. It is the purpose of this short paper to discuss briefly the aspects of conventional procedure most essential to securing a serviceable repair.

The importance of these procedures will be more evident if consideration is first given to the qualifications of a good serviceable repair or restoration:

1. It must be thoroughly and permanently bonded to the older concrete.

2. It must be sufficiently impermeable to prevent moisture reaching underlying older concrete.

3. It must, after drying, be free of shrinkage cracks through which water could reach the supporting concrete.

4. It must be resistant to freezing where this is a factor in weathering.

5. It must be of good matching appearance in relation to surrounding concrete surfaces.

BONDING

The first step in obtaining a well bonded repair is preparation of the older concrete which is to receive it. This is an operation that is easily slighted and, unless this becomes a firmly established check point for inspection, many a rock pocket on new work will be mortar filled or plastered over without removal. On restoration work, it is easy to stop too soon in removal of old concrete. Such short cut performances are prevented only by establishing a firm policy that no new material will be placed in repairs until preparation for it has been inspected and approved.

On either new work or restoration work, preparation involves:

1. Removal of all unsound or damaged concrete.
2. Shaping the excavation in such a way as to make the replaced material most secure.
3. Cleaning and possibly premoistening surfaces where good bond is desired.

When chipping out imperfect new concrete, it is readily evident when sound concrete is reached. If there is doubt, the excavation should proceed until no further question exists. This also applies to removal of weathered concrete, but it is not always as easy to be sure that enough has been removed. There is often a "twilight zone" that looks reasonably good and is hard to break away, but which may be partially deteriorated and will continue to disintegrate in a few more years of freezing and thawing, particularly if water gets to it, as it may, through cracks or by general absorption in wet weather.

Since very inferior concrete seems amply adequate when it comes to digging it out, tools for this purpose should be powerful and sharp. Air driven equipment is commonly satisfactory. Bits should be gad points because chisels crush and destroy areas of the surfaces, making them less likely to bond well than the unmarred break planes left by a gad point. If the work goes through a wall or other structural unit, concrete should be removed from both sides to avoid spalling which usually will be detrimental to the desired shape of the hole.

Shaping of the hole or edges of the area to be replaced depends on the method by which it should be restored, but this in turn depends on the size and general shape of the imperfection. If the hole is narrow or deep in relation to one surface dimension, as for a slot along a crack or a form-tie cone hole, and dry packed mortar is to be used for filling it, the edges should be sharp and undercut sufficiently to lock in the repair. A saw-tooth bit is effective and efficient for cutting slots and a drill bit or a stub of wire rope will score the inside of cone holes suitably.

If the hole or area is wide in relation to its depth, it may be filled by some form of pneumatically placed mortar or shotcrete (when its greatest

Elected a vice-president of ACI in 1959, **Lewis H. Tuthill** is concrete engineer with the California Department of Water Resources, Sacramento. Extremely active in ACI activities, he has been a member of the Board of Direction, Technical Activities Committee, and a member of various technical and administrative committees. He is a past chairman of ACI Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete, and Committee 611, Inspection of Concrete.

Mr. Tuthill (with William A. Cordon) was awarded the 1956 Wason Medal for the most meritorious paper and he was awarded the ACI Construction Practice Award in 1945.

Mr. Tuthill assumed his present position in 1956; previously he had been associated with the Bureau of Reclamation in Denver for 17 years.

dimension is only a few feet or the appearance of cracks is unimportant). For this method of replacement, edges should be sloped outward to avoid inclusion of rebound. If instead, hand placed mortar is used, the edges should be kept sharp and straight and all portions of the area should be at least 1 in. deep.

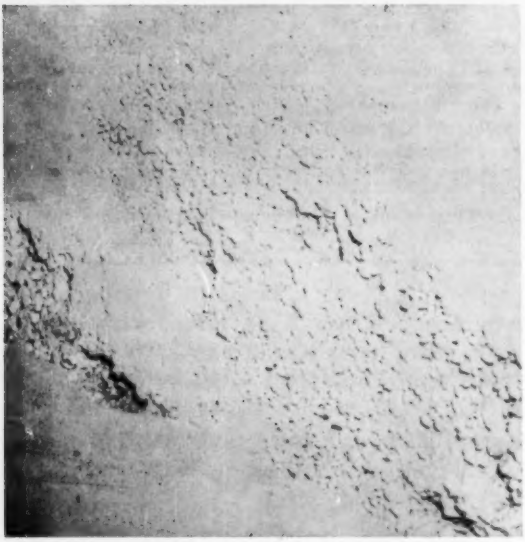
If a hole is both fairly wide and deep, or largely horizontal, it is best filled with concrete. For this method, when the hole is in a vertical plane, it must be shaped to provide:

1. Sharp edges at perimeters.
2. Straight edges at sides and top.
3. Inside faces generally normal to the formed surface, except at the top where it must slope up toward the chimney for filling, at about 3:1.
4. Keying as necessary to lock the replaced material firmly in place in plain concrete.
5. Excavation going at least 1 in. behind exposed bars where there is embedded reinforcing.
6. All interior corners in these and other repairs rounded with a radius of about 1 in. for best contact and compaction of the raw material and best watertightness.

When the repair is essentially horizontal, it is necessary only that the edges at the perimeter be sharp. A concrete saw is a good way to obtain good sharp edges.

Chipping and trimming operations leave considerable stone dust on contact surfaces of the repair. Other dirt and contamination may reach these surfaces before the replacement is made. Good bonding depends

If inspection of newly stripped surfaces is not prompt, imperfections may be covered by a quick plaster job, and this seldom forms a good bond



With firm, thorough inspection of concrete handling, placing, and vibration, such rock pockets as this would be avoided and no repair would be necessary

primarily on these surfaces being completely clean and free of everything except the residual concrete. Thorough scrubbing with a wire brush is good, but sand blasting is surer and, therefore, preferable. Most of the dust can be washed away with chipping debris when that work is completed, but sand blasting should be scheduled just before the replacement.

Heretofore, instructions on bonding fresh concrete or mortar mixtures to older concrete usually emphasized that the surface of the older concrete should be kept wet for several hours, preferably overnight, prior to placing the new mortar or concrete against it. More recent observation has indicated that at least equally good bond, and possibly more permanent bond, is obtained when joint surfaces are approaching dryness when the new material is placed on them. Apparently, the effort formerly expended on moistening before the repair can better be transferred to more water curing afterward. On this basis, the preparatory sandblasting, if wet, should be done early enough for the surface to become dry again. Dry sand blast is harmful to workmen, and a final washing is needed anyway.

It should be noted that moist surfaces are still considered necessary to good bonding of dry pack repairs and the filling of cone holes. Such moist surfaces should be dusted lightly with dry cement which will absorb surplus moisture. Painting with neat cement grout tends to wet and soften the small volume of dry pack material required, and to cause it to shrink more on drying.

For bonding, contact surfaces against both hand placed mortar and concrete replacements must be carefully treated before placing the new

material. The best method is to shoot on a thin layer of mortar pneumatically. The impact of a thin dash coat is desirable if pneumatic means are not feasible. Wire brushing or rubbing in the mortar by hand (in a rubber glove) is effective. The mortar should be proportioned as in the replacement mix and soft enough to work well into the surface, building it up not more than 1/8 in. at most. Coarser sand particles that are unembedded can be swept away before placing the new material.

Best assurance of bond in small areas is obtained where it is appropriate to use mortar placed by pneumatic or shotcrete methods. Unfortunately, these methods do not produce satisfactory results in all respects in some cases (which will be discussed later), and, therefore, are not considered first choice for repair of larger scope, despite this advantage of bonding in relatively small areas.

IMPERMEABILITY AND FREEDOM FROM SHRINKAGE CRACKING

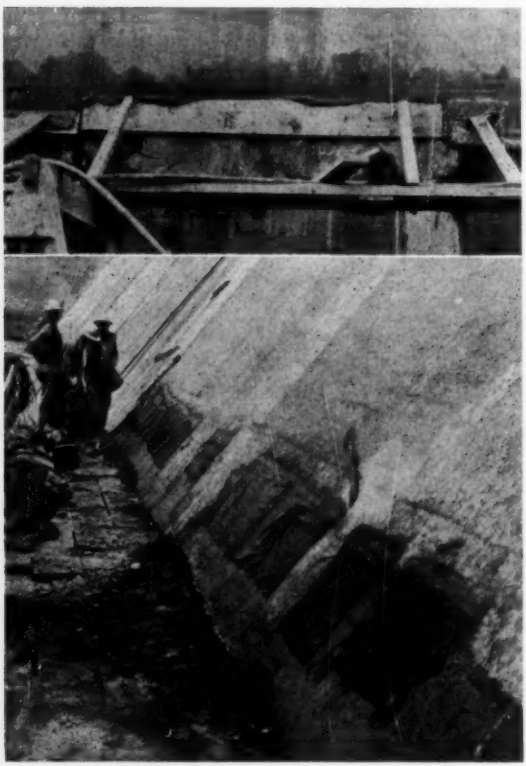
Impermeability of the repair concrete is not likely to be as important in new work as it is in the restoration of concrete deteriorated by weathering. Today, with wide use and understanding of the role of entrained air in improving the resistance of concrete to many cycles of freezing and thawing, most important new work is thus protected and made much more durable by its use. But many older concretes, which exhibit the lack of entrained air in their need for restoration, face a continuing hazard from weathering, and thus present a serious problem in their restoration. Unless moisture can be kept from it, underlying concrete

Restoration of weathered surfaces is unlikely to stop further weathering, unless the old concrete is protected from further wetting by the new work

subject to freezing and thawing will continue to deteriorate, with ultimate failure as a certain consequence. Restoration methods and materials, then, must produce a repair that is impermeable in itself and free from shrinkage cracks which will convey moisture to the inherently nondurable underlying old concrete.

Restoration of weathered walls, spillways, piers, bridges, and particularly weathered portions of hydraulic structures, may be more safely and permanently restored if, after deep removal of the old concrete, a self-sustaining layer of reinforced, air-entrained concrete is anchored to the old concrete. With properly designed and located construction and contraction joints with flexible waterstops, such a wall surface can be made reasonably watertight. Moreover, its probable greater thickness, although not preventing low temperatures in the old concrete, will reduce materially the frequency with which its temperature rises above and falls below the freezing point.

Pneumatically placed mortar or some form of shotcrete are commonly suggested and often used for restoration work. Despite its serious limitations in many cases and the fact that it is by no means inexpensive, it is



New material in repairs, usually being comparatively shallow, may be quickly damaged by drying if positive curing protection is neglected. If not fully water cured, at least a day or two of moist curing should precede application of a sealing compound. Top — Wet burlap mats held firmly against new repair by braces from scaffold. Bottom — Wet burlap mats held on a slope by temporary plaster along top edge and at other points

often chosen in preference to a thorough, deep-seated concrete replacement because of its convenience, facility, and speed.

In facing such a decision, it would be extremely worthwhile to read the ACI Standard "Recommended Practice for the Application of Mortar by Pneumatic Pressure (ACI 805-51)"* and Committee 805's valuable closure to the discussion.† The implications, as well as some of the statements of this standard, concerning the disadvantages and inspection of mortar placement by pneumatic methods, should be most carefully considered for restorations in freezing climates, particularly of hydraulic structures. Some are most pertinent to considerations of impermeability and minimum shrinkage cracking as important factors in arresting continuing deterioration under the repair. For instance:

1. "Usual experience has been that shotcrete is sufficiently porous so that coatings of it on structures exposed to weather or water will allow enough moisture to filter through to the contact between the shotcrete and the original material to permit freezing there, which in turn causes the coating to break its bond, although the coating itself may remain intact for years after it has been loosened."†

2. "Shotcrete is more subject to shrinkage cracks than concrete."*

3. "Some carefully obtained data available to the committee shows a range of 0.07 to 0.12 percent long-time drying shrinkage for shotcrete of various aggregate gradings and amounts of mixing water; while McMillan's, *Basic Principles of Concrete Making* and the USBR 1949 *Concrete Manual*, show a range for concrete of from 0.04 to 0.05 percent."†

4. "... in addition to continuous inspection during placing, all flat surfaces may be sounded with a hammer to detect inadvertent inclusion of rebound or the presence of hollow spots."*

5. "... both its structural value and the texture of the finished surface are greatly influenced by the skill of the workmen."*

In view of these well considered warnings by the committee, great care must be exercised in the choice and execution of restoration methods if long range serviceability is to be obtained in addition to an improvement in appearance. Whatever the method, good results can be expected only if the work is done correctly under an inspection that is thorough, convinced of the necessity of procedural requirements specified, dedicated to securing the intended results, and fully supported by its management in its efforts to do so.

RESISTANCE TO WEATHERING

Most aggressive effects of weathering are from repeated cycles of freezing and thawing. In mortar and concrete, we can obtain greatly improved resistance to this by means of air entrainment. Shotcrete apparently is sufficiently porous that no report was found of its disintegration by freezing. Both concrete and shotcrete are subject to some

*ACI JOURNAL, V. 22, No. 9, May 1951 (*Proceedings* V. 47), pp. 709-720.
†ACI JOURNAL, V. 23, No. 4, Part 2, Dec. 1951 (*Proceedings* V. 47), pp. 720-1 – 720-8.

Even good, sound repairs will be unsightly if their appearance is not carefully made to blend unobtrusively with surrounding surfaces

deterioration from expansion and contraction from wetting and drying. The resultant serious cracking of shotcrete has been discussed.

From this, it is evident, whatever the structure or the method used in repairing it, that weathering will be less severe, the drier it is practical to maintain the structure. Obviously, for some structures, this is impossible. For many others, much more favorable service conditions can be assured if good drainage and perhaps certain minimal shelter is provided. For many years, some of the prominent maintenance departments have protected exposed restoration and new work with a linseed oil and paint treatment.* Where it has been maintained, it has clearly been beneficial as has been observed on reclamation, as well as highway structures in the northwest. It is likely, however, that among the silicone treatments now available that there are waterproofing systems which are at least as effective, but less costly in that fewer initial and retreatment coats are necessary, and that the conditions of application are much less restrictive for good results. Silicone treatments should be further investigated as part of a restoration operation because continued serviceability is doubly assured by a good waterproofing treatment of critically exposed tops of walls, ledges, sills, curbs, corners, and other such parts. Perhaps, if shotcrete could be finished so that it could be treated efficiently, and the treatment was maintained, its porosity and cracking could be effectively sealed, thus neutralizing its most serious limitations for rehabilitation work.

In repairing new work, job cements and aggregates are used without question, and properly so, because they are presumably the best available, or at least satisfactory. A different approach is necessary in selecting materials for restoration of deteriorated structures. Particularly, a choice of aggregates should be made on the basis of any superiority exhibited in reducing the amount of shrinkage, whether mortar, concrete, or shotcrete repair methods are used. Significant differences have been reported

*Paxson, G. S., "Maintenance and Repair of Concrete Bridges on the Oregon Highway System," ACI JOURNAL, V. 17, No. 2, Nov. 1945 (*Proceedings* V. 42), pp. 105-116.

which indicate that such inquiry, and possibly comparative testing, is worthwhile.* Comparative concrete mix tests will reveal relative unit water requirement of different aggregates. Drying shrinkage and resultant cracking is usually in an increasing proportion to the water requirement. Sometimes, the content of certain clays may increase shrinkage more than their effect on water requirement might indicate. Sand passing the California Division of Highways "sand equivalent" test with a value in excess of 80 is unlikely thus to contribute excessively to shrinkage.

The foregoing effects on shrinkage are only indirect factors affecting resistance to weathering. Sometimes, there are inherent aspects of certain aggregates that are significant in this regard. Their comparative effects can often be concluded from observation of service performance, and where comparative freezing and thawing tests are feasible, the results may be most revealing.

APPEARANCE

One of the prime reasons repairs are made in new or old concrete is to make it more presentable. On new work particularly, and also on restoration work where only part of the visible surface is treated, appearance of repairs will be offensive, unless they are carefully constructed and finished to blend unobstrusively with surrounding untouched surfaces. Where an over-all restoration is performed, good appearance depends primarily on uniformity of texture and color, regardless of what the old concrete may have looked like. It is much more difficult to match a surrounding surface with complete obscurity than it is to create an over-all pleasing effect.

Perhaps the greatest problem is with small repairs on new work: a cone hole to fill; an offset to bevel; a rough spot to make smoother. Although these can be treated with a minimum of labor and thereby make them quite inoffensive, too often they are made conspicuous, as if the imperfection had been much larger, by the overenthusiastic efforts of the workmen. The cone hole could be neatly tamped full using a lean, stiff, lightcolored mortar, finished only with a piece of wood struck with a hammer instead of wiping it full with a mortar that is too rich, dark, and wet, troweling it with steel, and making a mess a foot in diameter. The offset could be lightly trimmed with a good stone held at a bevel so that nothing would be touched except the sharp, shadow-throwing edge, instead of scratching up a strip several inches wide and then rubbing a too rich, dark mortar in it to smooth it up, and in the process leaving an eye-sore for all time. The rough spot should be examined critically in the

*Discussion by Crom, J. M., of "Recommended Practice for the Application of Mortar by Pneumatic Pressure (ACI 805-51)," ACI JOURNAL, V. 23, No. 4, Part 2, Dec. 1951 (*Proceedings* V. 47), pp. 720-1 — 720-8.

light of what it may look like if worked on, and often left alone. If it is too unacceptable, a lean, light-colored mortar might be put under the stone while it is carefully trimmed down. Light-colored repair mortars are obtained by using the proportion of white cement found to make the mortar blend best on drying with surrounding surfaces.

Only wood or stone should be used to finish repairs on formed surfaces. Steel should never be used because it makes a dark, conspicuous texture even where steel forms have been used. An effort should be made to "print" form marks on unformed repairs of formed surfaces by continuing board marks and impressing a certain grain by placing a piece of sharply grained form board on it and striking the board with a hammer. A smooth board and no lines can be used in steel formed areas.

Repairs more than a square foot in area, in either new or restoration work, if they are to be sightly, should have boundary lines that are straight and preferably horizontal and vertical or parallel to nearby edges or other distinguishing lines in the structure. These edges are best cut with the diamond saw and, if there is much to do, will be less expensive than careful hand cutting. It will also make a better shaped edge for security of the repair because it is easier to undercut a little with the saw.

SUMMARY

Sound and unobtrusive repairs can be made by conventional methods, if known, proven procedures are carefully followed in complete detail.

In this short discussion of conventional methods of concrete repair, it has been possible only to call attention to certain critical points which are especially vital to good serviceable results. For a complete coverage of detail procedure essential in obtaining serviceable repairs, it is recommended that reference be made to Chapter VII of the generally available Bureau of Reclamation *Concrete Manual*. These procedures were independently developed from experience, but the wide and considerable background behind these practices becomes more apparent if review is made of M. Hirschthal's, "Maintenance and Repair of Concrete Structures in Railroad Construction," ACI JOURNAL, V. 8, No. 3, Jan.-Feb. 1937 (Proceedings V. 33), especially pp. 276-278, and R. B. Young's, "Concrete: Its Maintenance and Repair," ACI JOURNAL, V. 8, No. 4, Mar.-Apr. 1937 (Proceedings V. 33), pp. 367-393. Probably the most important injunction of all is phrased by Hirschthal when he says:

"Strict adherence to all parts of specifications for repairs is essential."

Presented at the ACI 56th annual convention, New York, N.Y., Mar. 17, 1960. Title No. 57-6 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 2, Aug. 1960 (Proceedings V. 57). Separate prints are available at 50 cents each.

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Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1960, for publication in the March 1961 JOURNAL

Title No. 57-7

From ACI Committee 201 Symposium on
Restoration of Deteriorated Concrete

Repair of Concrete Pavement

By EARL J. FELT

Patching of distressed areas of concrete pavements with bonded concrete can be accomplished successfully if established principles are followed. Most important is a clean, sound, old concrete surface. In addition, high quality grout and concrete, and first class workmanship are essential. Suggestions are given for cleaning and preparing the old surface, for grouting, and for placing concrete.

■ REPAIR OF DISTRESSED AREAS of concrete pavement may be accomplished either by full-depth patching or by bonded patching. As full-depth patching offers no unusual problems, this paper will be confined to a discussion of methods of patching with relatively thin layers of bonded concrete.

The question most commonly raised in connection with patching with a thin layer of concrete is whether or not the new concrete can be made to bond securely and permanently to the base. That it can be done with success is known, for there are many examples where long lasting bond has been obtained. For instance, a street in Toledo, Ohio (Fig. 1) damaged by freezing when paved in 1912, was resurfaced with a 1- to 2-in. layer of concrete in 1913. Cores removed 40 years later from one of the better areas of the street showed that good bond still existed. In Savannah, Mo., a concrete street resurfaced in 1914 with 2 to 3 in. of concrete was still in service after more than 40 years. A patch of bonded concrete less than 1 in. thick placed in 1945 on a part of a bridge sidewalk in Dublin, Ohio, where severe scaling had occurred, was still well bonded 15 years later (Fig. 2). Other examples, illustrated by Fig. 3, can be cited where good bond has existed for many years.

The question then is not whether bond can be obtained, but rather "what procedures should be followed to insure good bond?" Considerable work has been done to answer this question, and many different methods have been proposed for bonding new and old concretes. For this reason a comprehensive laboratory and field study having to do with bonding new and old concrete has been conducted during the past several years by the Portland Cement Association.[1,2] This work has been correlated with that done by others.[3]*

*Reference 3 is an annotated bibliography of 175 articles pertaining to resurfacing and patching concrete pavements with concrete.

STUDIES OF BONDING

A laboratory study evaluated factors that might influence bond between new and old pavement-quality concretes, such as smooth and rough surfaces; damp and dry surfaces; sand-cement and neat cement grouts; concrete mix proportions; and methods of placing the new concrete, including surface and internal vibration, vacuum processing, mechanical float consolidation, and pneumatic pressure application. These factors were evaluated by a test in which a bonded layer of new concrete was sheared from an old concrete base. The investigation included tests of base concrete approximately 25 years old, although in preliminary tests many variables were evaluated using laboratory cast base concrete which was only a few weeks old.

A relatively smooth surface was used on most laboratory cast base specimens to permit a better evaluation of variables other than surface roughness. Most of these specimens were given an acid treatment which involved wetting the concrete surface with water followed by a uniform application of commercial hydrochloric acid (18 deg B, 28 percent HCl) at the rate of 1 gal. to about 8 sq yd of surfacing. The residue was carefully removed by washing and vigorous brushing. This treatment was used to assure removal of laitance which might otherwise confuse the test results and make it difficult to evaluate the test variables. The surface of the 25 year old concrete was prepared in several ways to study the effectiveness of various cleaning treatments.

Fig. 1—Thin resurface placed in 1913 on frost-damaged concrete

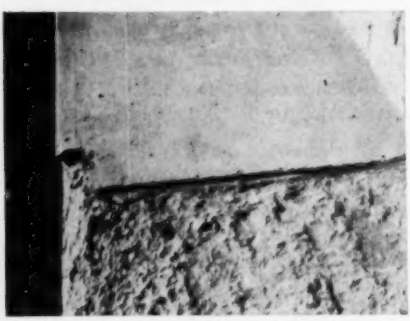

Fig. 2—Thin patch on bridge sidewalk. Non-air-entrained concrete is in foreground

In the first group of tests, base specimens 9½ in. square and 7 in. thick were cast of non-air-entrained 6-sack concrete. The prisms were cured in the fog room for 21 days followed by at least 5 days at 73 F and 50 percent relative humidity. The surfaces of most blocks were then treated with hydrochloric acid as just mentioned, or roughened with the Tennant machine* as noted in Table 1.

Specimens to be surfaced "dry" were conditioned at 100 F and 30 percent relative humidity for 2 days, and those to be surfaced

*The Tennant machine manufactured by G. H. Tennant Co., Minneapolis, utilizes rapidly revolving hardened steel gear-shaped cutters for chipping the old concrete surface.

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"damp" were conditioned in the fog room for 2 days. The surfaces of some blocks were then coated with a neat cement grout or with a 1:1 stand-cement grout having a water-cement ratio of about 5.5 gal. per sack; other blocks were not grouted. A 2-in. layer of non-air-entrained 6-sack concrete was then placed on the blocks. After curing 14 days in the fog room followed by 14 days at 73 F and 50 percent relative humidity, the top layer was sheared from the base as shown in Fig. 4.

In a second group of tests, concrete base specimens 16 x 40 x 7 in. were cast and cured as described for the 9½-in. base blocks. The surfaces of most of these specimens were treated with HCl, but as indicated in Table 2, some surfaces were sandblasted. The specimens were surfaced with 2 in. of air-entrained concrete, frequently two or four at a time (Fig. 5) which permitted a study of concrete placement methods that could not be readily investigated using the 9½-in. blocks. The specimens were removed from the concreting setup by sawing around them; they were then cured 14 days in the fog room, after which four 8-in. prisms were sawed and four 8-in. diameter cores were drilled from them. These latter specimens were conditioned 14 additional days at 73 F and 50 percent relative humidity before shear testing.

In a third group of tests, 16 x 40-in. specimens sawed from 25 year old concrete pavement slabs were surfaced with 2 in. of air-entrained concrete to evaluate various methods of cleaning the concrete to develop good bond. Methods including washing with acids and other liquids, and cleaning mechanically with a Tennant machine, with electric chisels, and by sandblasting. Methods of grouting and placing the resurfacing concrete found to be practical and effective in the previous tests were used in resurfacing. These slabs, resurfaced in the air-dry condition, were then cured and sawed into prisms or were cored as previously described for the laboratory cast slab specimens.

Fig. 3—Full lane thin patch, repair of both sides of spalled transverse joint

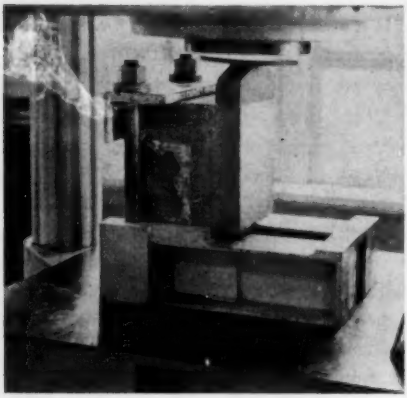

Fig. 4—Testing prism in shear to measure bond

Fig. 5—Four specimens ready for resurfacing. Two outer specimens are dry; two middle specimens are damp; one-half of two specimens covered with grout

Concurrently with the laboratory study, several pilot field projects were built to investigate the effect on bond of cleaning the old surface, wetting the surface, grouting, and concrete placement procedures.

RESULTS OF STUDIES

The data on laboratory cast base specimens presented in Tables 1 and 2 show that good bond between new and old pavement-quality concretes was obtained by several different procedures.* Although good bond was obtained with both damp and dry base concrete, the data, when averaged as in Fig. 6, show that best bond was generally developed with a dry base concrete (except when a zero-slump resurfacing was used). This appears contrary to the generally accepted practice of keeping the old surface wet for at least several hours before placing the new course. However, a number of other investigators[4-7] have also found that a dry surface develops better bond than a damp surface.

Test results show that a grout bonding layer generally aided bond. The grout wets the surface uniformly, displaces air films, and incorporates any loose material still on the old surface. The data indicate little difference between sand-cement grout, neat cement grout, retempered grout, grout containing $CaCl_2$, or a grout formed by spreading dry cement on a wet surface.

Good bond was obtained with surface vibrated concrete, with internally vibrated concrete, with shotcrete, and with vacuum processed concrete; also with no-slump concrete consolidated with a mechanical float immediately after placing on a damp or on a dry, grouted surface;

*Additional data and discussion are presented in Reference 1.

and with normal 2-in. slump concrete that was permitted to set in place for 2 or 3 hr before final consolidation with a mechanical float.

These results explain why successful bond has been reported by many investigators using entirely different methods.

Test results on 25 year old base concrete, given in Table 3, show that the most important single factor that governs bond is the surface condition of the old concrete. It must be sound, durable, and clean. Unsound concrete must be removed mechanically before placing new concrete. Chipping the old surface with an electric chisel was helpful, but the most effective tool for mechanical cleaning and for removing unsound concrete was the Tennant machine.

After removal of unsound concrete from the surface, many of the 25 year old base specimens developed bond strength greater than the strength of the old concrete. This resulted in shear failures about ¼ to 1 in. into old concrete.

Sandblasting the old surface was not as effective in these tests as it was in the tests reported in Table 2. The sandblasted surface appeared clean but all projections seemed to be rounded, which may have had some effect on bond.

Cleaning and slight roughening of an old, sound surface with hydrochloric acid developed acceptable bond in the range of 300 to 500 psi.

TABLE 1 — EFFECT OF SURFACE TREATMENT AND VARIOUS TYPES OF GROUT ON BOND*

Surface treatment given base specimen, and description of special grout	Bond strength, psi					
	Damp base			Dry base		
	Grout		No grout	Grout		No grout
	1:1	Neat		1:1	Neat	
None (little laitance effect)	216	239	254	264	253	308
None (wood-float finish, laitance effect)	136		152	153		140
HCl	339	232	295	625	408	353
HCl not flushed and brushed			93			106
Tennant, shallow waffle	415	345	336	388	410	543
Tennant, deep waffle					500	
HCl, 2 percent $CaCl_2$ in grout	295	229		452	503	
HCl, retempered grout†	620	317		517	403	
HCl, cement dust grout		547			519	
HCl, grout sprayed at 26 psi		392			564	
HCl, grout on surface 1½ hr before resurfacing	353	259		440	437	

*9½ x 9½-in. prisms surfaced with 2 in. of 6-sack concrete consolidated by internal vibration. Data are average of at least three tests.

†Grout was mixed, then permitted to set undisturbed 1 hr, then remixed.

Some roughness is desirable but extremely rough surfaces are not required. Although Clemmer[8] and Ungar and Barentzen[9] have found by experience that hydrochloric acid is an effective cleaning agent, the literature indicates that there has been disagreement among other researchers as to the value of acid cleaning. Our tests showed that the HCl treatment appeared to damage slightly the surface of the concrete and leave a thin defective layer. However, the treatment greatly aided bond, *provided the residue and partially loosened sand grains were completely removed by washing and vigorous brushing.* If the residue was not completely removed, bond was adversely affected.

The bond tests previously discussed were made on specimens 28 days old. Therefore, several tests were made to determine the rate at which bond strength increased with time of curing up to an age of 28 days. Two 25 year old 16 x 40-in. slabs were cleaned and etched with hydrochloric acid, coated with a 1:1 grout, and resurfaced with 2 in. of 7-sack

TABLE 2 — EFFECT OF SURFACE TREATMENT, CONCRETE CONSISTENCY, AND PLACEMENT PROCEDURE ON BOND*

Method of placement, type of resurface concrete	Bond strength, psi				Type of test specimen
	Damp base		Dry base		
	1:1 grout	No grout	1:1 grout	No grout	
Surface vibration, 6 sack per cu yd, 3-in. slump, 6 percent air	297	194	521	167	8-in. prisms
Internal spud vibration, 6 sack per cu yd, 3½-in. slump, 4½ percent air			363	212	8-in. prisms
Compactor float, after hand tamping, 7 sack per cu yd, 0-in. slump	579 640	397 468	598 687	0† 0†	8-in. prisms 8-in. cores
Internal spud vibration, compactor float, 2 hr later, 7 sack per cu yd, 2½-in. slump, 4 percent air	437 620	350 430	659 816	449 657	8-in. prisms 8-in. cores
Shotcrete 1:3 mix, specimen vertical	289 124‡	434 350	428 151‡	301 553	8-in. prisms 8-in. cores
Shotcrete 1:3 mix, specimen horizontal	251 135	354 214	536 373	590 577	8-in. prisms 8-in. cores
Shotcrete 1:3 grout instead of 1:1 grout, resurface mix 6 sack per cu yd, 1½-in. slump, 1½ percent air, internal vibration	477 820	269 235	574 820	481 472	8-in. prisms 8-in. cores
Surface vibration followed by vacuum process, 6 sack per cu yd, 3-in. slump, 6 percent air	189	166	456	272	8-in. prisms
Internal spud vibration, 6 sack per cu yd, air-entrained; repeat of second test reported above			475		8-in. prisms
Same as immediately above, but surface was sandblasted			391		8-in. prisms
Repeat of test immediately above			468		8-in. prisms
Same as immediately above, but sandblasted surface was treated with HCl			463		8-in. prisms

*16 x 40-in. slabs treated with HCl (except as noted) and surfaced with 2 in. of concrete. Data are average of tests on two 8-in. cores or on two 8-in. prisms cut from the slabs, except on four prisms where no data are shown for cores.

†Resurface came off during sawing and coring.

‡Sand pockets apparent in concrete.

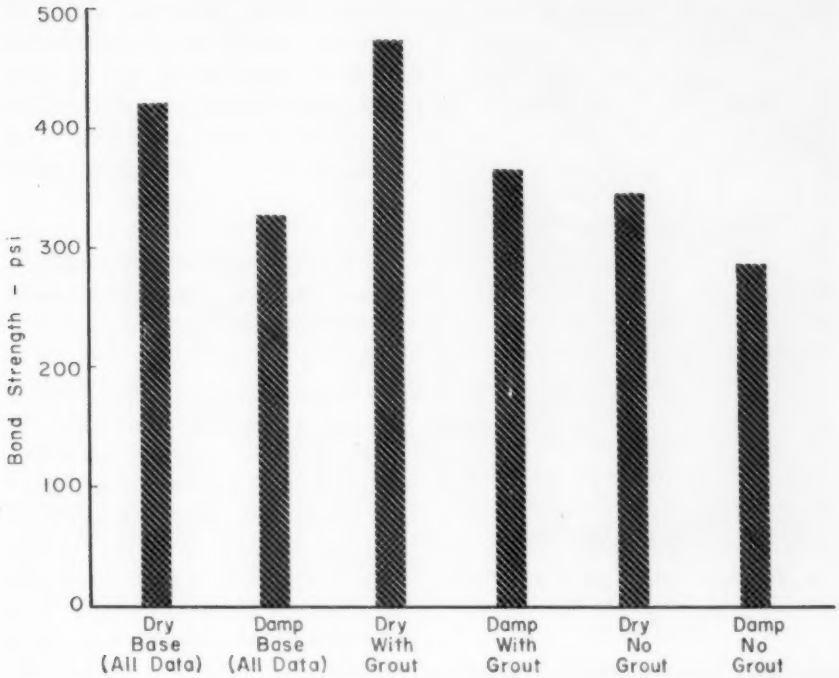

Fig. 6—Effect of moisture condition of base and effect of grout on bond (averages of data in Tables 1 and 2)

concrete placed by internal vibration. The concrete on one specimen contained 2 percent $CaCl_2$ by weight of cement. In addition, several 9½-in. prisms of laboratory cast concrete were surfaced using the same concretes. Specimens were tested in shear at the ages shown in Table 4. According to these data, bond strength of more than 300 psi developed rapidly.

The laboratory findings were corroborated by the field studies. Good bond was obtained under several different construction practices with both dry and damp base concrete. Again, the data showed that permanent and strong bond was dependent principally upon the condition of the old concrete surface. Several cores, when sheared in laboratory tests, broke in the old concrete (Fig. 7) rather than at the bond surface.

TYPES OF DEFECTS TO BE REPAIRED

Probably the most common defect in present day concrete pavements is spalling at the joints, frequently attributable to poor joint design or to improper construction. If air-entrained concrete has been used, the spalls are hard, durable concrete (Fig. 8), and the shearing surface

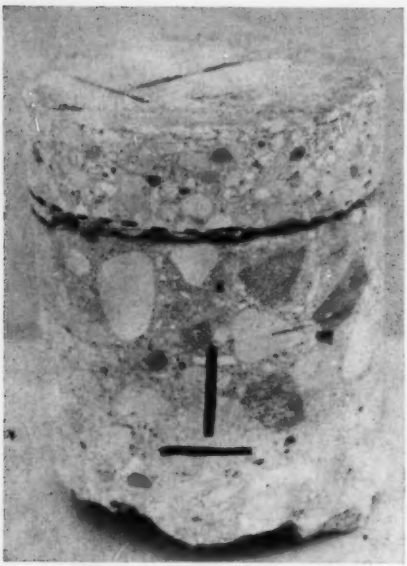

Fig. 7—Shear failure in old concrete base. Dotted line shows bonding interface

makes a good base for a bonded concrete patch. Many pavements made of non-air-entrained concrete still exist, however, and spalling at joints is also associated with concrete scaling and disintegration (Fig. 9). Here the base concrete may not be sound and durable, and considerable removal of old concrete will be required before patching with bonded concrete.

Scaling of concrete pavements has been practically eliminated by the use of air-entrained concrete; however, local scaled areas which require patching still develop on old pavements built of non-air-entrained concrete (Fig. 10). In this type of repair, complete removal of all concrete which has scaled or is otherwise defective is absolutely necessary before patching. In extreme cases, full depth removal of the old concrete may be required.

TABLE 3 — EFFECT OF SURFACE TREATMENT OF OLD CONCRETE ON BOND*

Method of preparing old surface base slabs in air-dry condition	Bond strength, psi	
	1:1 grout	No grout
HCl	301	228
HCl, internal vibration	363	212
Glacial acetic acid	52	68
Brushed and washed with water	21	4
Sandblasted, torpedo sand,shotcrete gun	102	88
Sandblasted, silica sand, commercial gun	48	64
Tennant machine,shallow waffle	283	326
Electric chisel, light chipping	163	162
†HCl, 7 sack per cu yd concrete internally vibrated	525	327
HCl, 11 sack per cu yd concrete internally vibrated	425	418
HCl, 7 sack per cu yd concrete (scaled but sound base)	358	
Tennant machine, 7 sack per cu yd (scaled but sound base)	407	

*16 x 40-in. slabs sawed from 25 year old pavement, surfaced with 2 in. of 6-sack air-entrained concrete consolidated by surface vibration except as noted. Data are average of tests on four 8-in. prisms.

†Base concrete for tests above this entry was from different source and was weaker than base concrete for remaining tests.

TABLE 4 — RATE OF GAIN IN BOND STRENGTH

Specimen identification	Bond strength, psi							
	Age of specimens when tested, days							
	Plain concrete				Concrete with 2 percent $CaCl_2$			
	3	5	7	28	2	4	6	28
25 year old base	318	302	300	484	345	347	384	656
Laboratory cast base	311	532	492	—	423	500	533	—

Other surface defects which require patching may occur when the old pavements are exposed to extremely high temperatures caused, for example, by a burning gasoline truck or airplane, or when newly placed pavements are exposed to freezing temperatures. The damaged concrete must be removed completely before patching or resurfacing.

FUNDAMENTALS OF REPAIR

First class workmanship and high quality materials are essential for successful patching with bonded concrete. Four fundamentals must be observed closely: (1) careful preparation of the old base concrete; (2) use of, and proper placement of, high quality grout and concrete; (3) proper jointing; and (4) proper curing.

Preparation of the old concrete

Adequate preparation to provide a clean, durable concrete base to which the new concrete will be bonded is the most important step in the bonding process. The area to be patched is usually outlined with a saw cut about ½ to 1 in. deep. Old blacktop patches, or areas of concrete which have been damaged by spalling, by burning, by freezing, or which are in the process of scaling, are completely removed mechanically with air hammers or with other equipment such as the Tennant machine. Scarifying or cutting (Fig. 11) is continued until sound, durable concrete is exposed over the entire area as shown in Fig. 12. In some cases

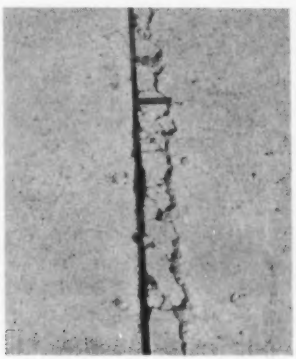

Fig. 8 — Spalled joint, air-entrained concrete

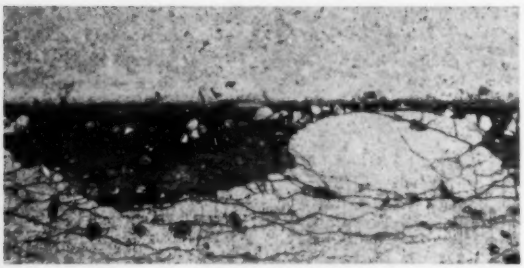

Fig. 9 — Spalled and distressed concrete at joint in non-air-entrained concrete

mechanical cleaning is supplemented with acid treatment (described below) to clean the surface thoroughly.

Mechanical removal of distressed concrete over large areas, as shown in Fig. 13, may be accomplished with the Tennant machine. When possible, the area is kept dry until all the dust can be removed with compressed air. Final cleaning is with water under high pressure to obtain a clean, sound surface as seen in Fig. 12. This should be done sufficiently ahead of concrete placement to permit the surface to become practically dry before patching.

If the old concrete base is sound and durable, mechanical removal of the old concrete surface may not be necessary. In such cases the surface is washed with water, followed by cleaning and etching with hydrochloric acid. If oil stains are present, the surface is scrubbed prior to the acid treatment, with a detergent such as sodium metasilicate with resin soap spread on the wet surface, at a rate of 1 to 2 lb per sq ft. The surface is then brushed and flushed with water until the residue is completely removed. Next, commercial hydrochloric acid (18 deg B, 28 percent HCl), applied at the rate of 1 gal. to 60 or 70 sq ft, is brushed over the wet concrete. Wetting dilutes the acid and facilitates uniform spreading. After the foaming action of the acid has ceased, the old concrete is thoroughly flushed and *vigorously brushed to remove par-*

Fig. 10—Scaled pavement, non-air-entrained concrete

Fig. 11—Removal of concrete at spalled joint

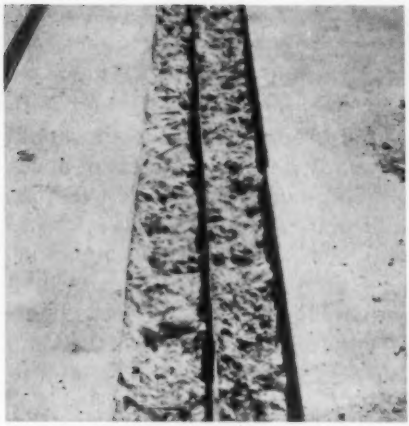

Fig. 12 (above)—Spalled joint area prepared for patching. Fig. 13 (right) —One lane of bridge deck prepared for resurfacing, other lane newly resurfaced

tially loosened sand and other residue. This brushing operation is essential and must be done carefully if good bond is to be obtained. A second treatment with acid may be necessary if an absolutely clean surface has not been obtained.

Grout, concrete, and consolidation

After final cleaning, when the surface of the concrete has just become dry, a 1:1 sand-cement grout or a neat cement grout is applied on the surface as shown in Fig. 14 and 15. A 1:1 sand-cement grout made with concrete sand scalped on a No. 8 sieve is easier to mix and handle than a neat cement grout and consequently has some advantage. The grout in a layer about ⅛ in. thick is carefully and thoroughly broomed into the surface to wet it uniformly, to displace air films, and to incorporate any loose particles still present. In hot weather, the surface of the old concrete may be dampened by light fogging, but the grout should not

Fig. 14—Grouting large surface

Fig. 15—Grouting spalled joint surface

be applied to an extremely wet surface or when hollows and rough areas contain free water. Such excess water may be removed with compressed air. The grout should have a water-cement ratio of about 5½ gal. per sack, which will produce a consistency suitable for use outdoors under most conditions. Sufficient grout may be mixed to provide material for about an hour. It should be remixed as it is being used, and if the relative humidity is low, slight amounts of water may have to be added. The grout should not be spread over such a distance that it will dry to a light gray color before placing of the concrete.

High quality, air-entrained concrete should be used for patching. Ordinarily, Type IA cement is used, but Type IIIA or either type cement with 2 percent calcium chloride may be used if high early strength is required. Thorough consolidation is necessary. If the patch is relatively small and the concrete can be hand tamped carefully, as shown in Fig. 16, the concrete may have a 1-in. slump. However, where the area to be patched is larger and reliance cannot be placed on hand compaction, the concrete should have a slump of about 2 in. and should

Fig. 16 (above)—Hand compaction of low-slump concrete. Fig. 17 (right)—Spud vibrator consolidation of plastic concrete

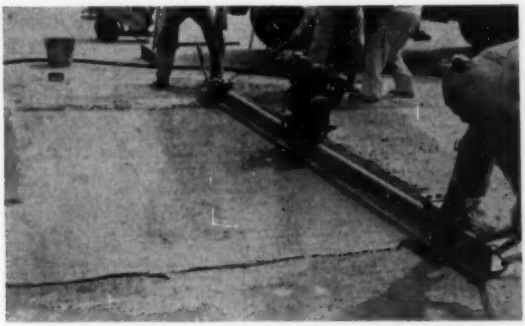

Fig. 18 — Vibratory-screed consolidation of plastic concrete

be consolidated either with a spud vibrator or with a vibratory screed as shown in Fig. 17 and 18.

The maximum aggregate size should not be greater than about one-half the depth of the patch, and the water-cement ratio of the concrete should be approximately 5 gal. per sack.

Jointing

Wherever a bonded patch extends over a joint or butts against a joint in the old pavement, a joint must be provided in the new concrete. Joints may be sawed in the hardened concrete if the time of sawing is controlled to prevent reflective cracking, or they may be tooled into the plastic concrete. Joint cutting may be avoided by installing metal inserts, or headers of wood, or expansion joint material. The inserts and headers are removed later by extraction or by sawing and are replaced with joint filler and sealer. In all cases the joints in the patches must be directly over the joints in the old pavement, they must be at least equal in width to the old joint, and they must extend the full depth of the patch. Proper jointing is essential if subsequent spalling is to be avoided. Jointing of each job is an individual matter and must be considered carefully.

Fig. 19 — Newly completed patch of spalled joint

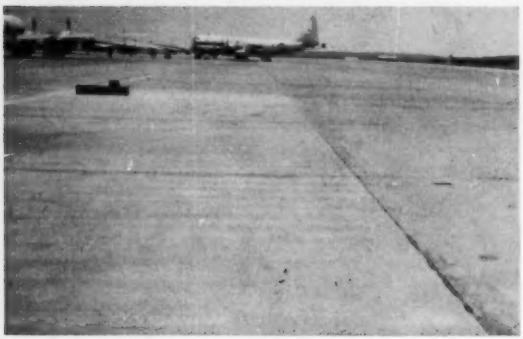

Fig. 20—Lane in left foreground is thin bonded repair of concrete damaged by freezing

Curing

Curing procedures should be designed to reduce temperature changes during the early life of the concrete when resulting volume changes may be detrimental to bond, and to prevent early drying. A combination of wet burlap covered with light colored vaporproof paper is suggested for the first 48 hr; then a white pigmented curing membrane should be applied. However, the use of either wet burlap or vaporproof paper alone, or of a white pigmented curing membrane, has proved satisfactory. On windy days when the relative humidity is low, an application of a light spray of water is suggested just before the curing materials are applied. The length of the curing period will depend upon the air temperature. Under normal summer conditions, the patch can be opened to traffic in about 3 days. If the patch must be opened sooner, high-early-strength concrete should be used. Some evidence is available which shows that in warm weather, low-slump concrete patches of spalled joints may be opened to traffic in 24 hr or less. A patch of spalled joint, just completed, is shown in Fig. 19.

A 1-in. bonded resurface about 700 x 25 ft is shown in Fig. 20. This resurface was placed to repair a 15-in. pavement which had been damaged to a depth of approximately ½ in. by freezing. The new surface is practically indistinguishable from the old concrete.

CONCLUSION

Experience has shown that the repair of concrete pavements with bonded concrete may be accomplished effectively. A clean, sound surface must be provided to insure success. If hydrochloric acid has been used to clean and etch the old concrete, the residue must be removed by vigorous brushing and flushing. High quality materials must be used and first class workmanship must be employed in grouting, placing and consolidating concrete, jointing, and curing. Since the bonded concrete layers may be relatively thin, the ambient weather conditions must be

considered. Special precautions must be taken to see that the slump of the concrete is compatible with the temperature, the size of the patch, the work crew, and the mechanical tools available. Curing operations should be planned so that the new concrete does not change temperature greatly during the first 24 hr, nor dry out during the first 3 days.

REFERENCES

1. Felt, E. J., "Resurfacing and Patching Concrete Pavement with Bonded Concrete," *Proceedings*, Highway Research Board, V. 35, 1956, pp. 444-469.

2. Westall, W. G., "Bonding Thin Concrete to Old," *Civil Engineering*, V. 28, No. 6, June 1958, pp. 406-409.

3. "Salvaging Old Pavements by Resurfacing," *Bibliography* No. 21, Highway Research Board, 1957.

4. Withey, M. O., "Bond of Vibrated Concrete," *Proceedings*, Highway Research Board, V. 16, 1936, pp. 193-206.

5. Thornton, J. C., "Relation Between Bond and the Surface Physics of Masonry Units — An Abstract," *ASTM Bulletin*, No. 190, May 1953, p. 45.

6. Waters, T., "A Study of the Tensile Strength of Concrete Across Construction Joints," *Magazine of Concrete Research* (London), V. 6, Dec. 1954, pp. 151-153; *Highway Research Abstracts*, Apr. 1955.

7. "Investigation of Methods of Preparing Horizontal Construction Joints in Concrete," *Technical Report* No. 6-518, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., July 1959.

8. Clemmer, H. F., "Patching Spalled Areas with Cement Mortar," *Roads and Streets*, V. 84, Mar. 1941, p. 46.

9. Unger, A. K., and Barentzen, W., "Concrete Roads Repaired by Application of a New Surface Course," *Cement* (Amsterdam), No. 5-6, 1951, pp. 98-100. Reviewed in ACI JOURNAL, V. 23, No. 4, Dec. 1951 (*Proceedings* V. 48), p. 361.

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From ACI Committee 201 Symposium on Restoration of Deteriorated Concrete

Title No. 57-8

Prepakt Method of Concrete Repair

By RAYMOND E. DAVIS

The prepakt method is used both in the restoration of old concrete and masonry structures and in certain types of new construction such as underwater work and work where the proper placement of conventional concrete would be difficult or impossible. The method is described, as are also the materials employed. The types of repairs for which the method is particularly well adapted are given. Mix proportioning methods are discussed. Methods of test for fresh prepakt grouts and hardened prepakt concrete are given. The properties of prepakt and conventional concrete are compared.

■ A TYPE OF CONCRETE AND A METHOD OF REPAIR employed in the restoration of masonry and concrete structures is known as "prepakt."* It was first used in the fall of 1938 for the strengthening of the concrete lining of Muir tunnel on the Santa Fe Railroad near Pittsburg, Calif. Since that time, extensive laboratory research concerned with this method has been carried out in the Engineering Materials Laboratory of the University of California, Berkeley, and in the concrete laboratories of the Corps of Engineers in Jackson, Miss., and the U. S. Bureau of Reclamation in Denver. The results of these and other investigations, coupled with experience in a wide variety of field applications, have brought the method along to its present state of development.

Prepakt is essentially a concrete of gap grading which is produced by first placing the coarse aggregate in the forms and thereafter filling the voids of the aggregate by pumping a sanded grout, sometimes referred to as intrusion mortar or prepakt grout. In the repairs of many old stone masonry and concrete structures by the prepakt method, neat cement grouting for the purpose of improving structural stability and reducing leakage is also carried out.

EARLY HISTORY

The story of the conception of the method and its early development is of some interest. It was first considered as a method of construction of dams. During the fall of 1937 a subcommittee of ACI Committee 207, Properties of Mass Concrete, of which I was then chairman, conducted a nationwide inspection of dams starting in the southeastern states. On our return from the Southeast, Franklin R. McMillan (ACI president,

*There are several methods where the aggregate is prepacked, but the materials and methods vary from those described in this paper.—EDITOR

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An ACI member since 1926, and now Honorary Member, he has been both president and vice-president of the Institute as well as a member of the Board of Direction and various administrative and technical committees. He received the Wason Medal in 1931 and 1934, the Construction Practice Award in 1948, and the Henry C. Turner Medal in 1952.

1936), Robert F. Blanks (ACI president, 1948), and myself (ACI president, 1942) stayed overnight in Washington. The subject of the evening's bull session was "cracks and leaky joints in dams, and what could we do about it." At this meeting I first sketched the scheme of building a dam without contraction or construction joints by the process of first placing all the coarse aggregates inside of forms or inside of a reinforced concrete shell. Then during the winter the aggregate mass was to be cooled with naturally cold water to near the freezing point. This was to be followed by grouting the mass as a continuous operation, using a sand-cement grout containing portland cement, pozzolan, and a water-reducing, set-retarding admixture. The scheme was so intriguing that after my return to Berkeley I started a research project concerned with the effect of size and grading of coarse aggregate, the grading of sand, and the composition of grout mixtures upon groutability of aggregate masses and properties of hardened grouted aggregate.

During the spring of 1938 Henry L. Kennedy (ACI president, 1953) visited our laboratory, and I told him about the scheme and the results of laboratory investigations. He encouraged me to apply for patent, which I did with the assistance of the patent department of his company, Dewey and Almy Chemical Co. The patent was granted in 1943.

Not long after Henry Kennedy's visit, there came to me Louis S. Wertz, whom I had never met before, then president of Durite Co. which was engaged exclusively in the repair of concrete structures. He was then doing a job for the Santa Fe Railroad in waterproofing a tunnel not far from Berkeley. The tunnel was leaking badly, and his job was to stop the leaks. This he was doing by pressure grouting with a grout mixture composed of portland cement, ground water-quenched blast furnace slag, and a waterproofing admixture, the principal constituent of which was a stearate. Patching was done through the use of pneumatically applied mortar. He cited some of his earlier jobs where under freezing and thawing conditions, failures had occurred due to scaling or failure in bond between mortar and old concrete after 2 or 3 years of service. He then showed me a sample of hardened grouted gravel which he had prepared by first filling a pipe with aggregate and then grouting under pressure. I told him that I thought this method might be expected to possess certain advantages over types of repair

or construction then available. This visit started a new round of investigations and a lifelong friendship.

Shortly thereafter the first grouted aggregate job was undertaken in Muir tunnel earlier referred to. For a length of about 300 ft a new highway crossed diagonally over the old tunnel. The highway cut left only about 3 ft of earth over the crown of the tunnel arch. This reduced the vertical load on the tunnel arch, and lateral earth pressures slowly shortened the span of the arch, raising the crown and producing longitudinal cracks at the crown and at the spring lines. To strengthen

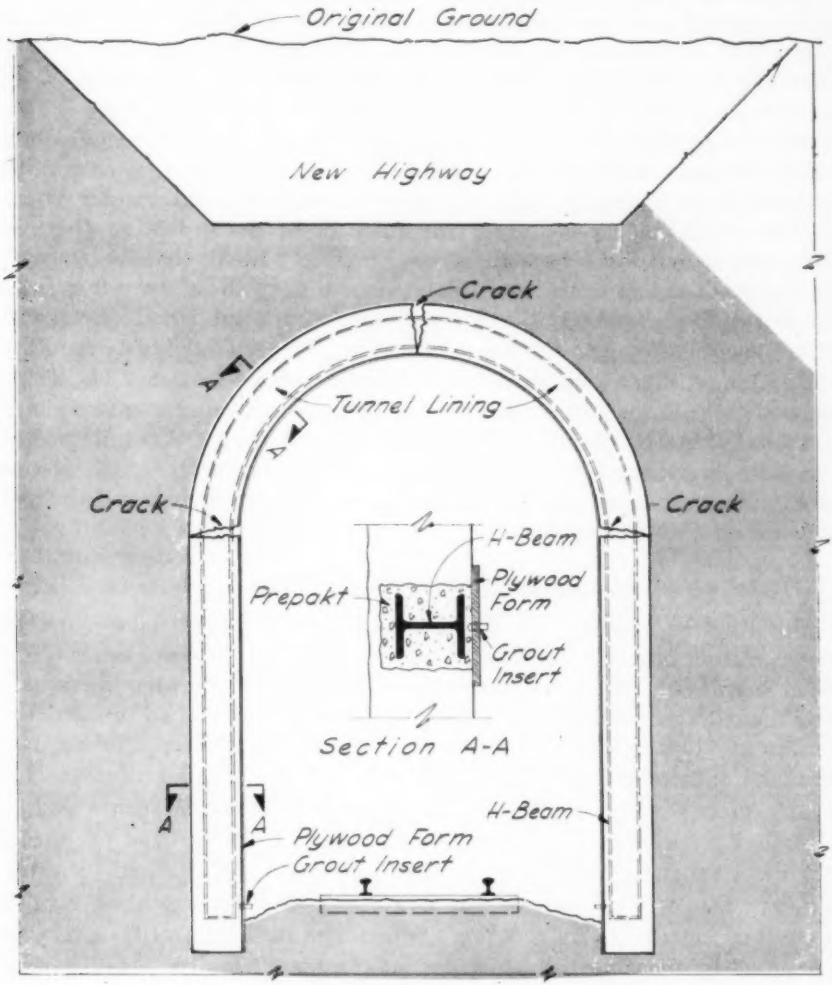

Fig. 1—Stabilizing tunnel lining

the arch and prevent further deflections, curved steel H-beams were fitted in transverse slots cut with jack hammers in the concrete tunnel lining as shown in Fig. 1. In most places the clearance between the flanges of the H-beams and the old concrete of the slot was not much more than an inch. After placing the curved H-beams in position, as the slots were formed with plywood, they were filled by hand with ⅜- to ¾-in. gravel which was vibrated into place with a small laboratory concrete vibrator. Then a grout of cream-like consistency, composed of a portland-pozzolan cement then available in the San Francisco Bay region, a sand all passing the No. 16 sieve, and the stearate admixture was pumped into each of the slots. This was done through inserts in the forms starting at the bottom and continuing up each side until it was evident from grout coming from the crack along the crown of the tunnel arch that the form had been filled.

Naturally there was a great deal of speculation as to the completeness of grout penetration of the voids in the gravel aggregate. To settle this question, a considerable number of cores were drilled. An examination of these cores and of the drill holes demonstrated that for all practical purposes the voids were completely filled; that the strength of the grouted aggregate was greater than that of the old concrete; and that the bond between the grouted aggregate and the old concrete was exceptionally good.

Sometime thereafter came the question of a name which would suitably distinguish this new kind of concrete from the conventional product. That is where Joe W. Kelly entered the picture. He is the Institute president for 1960, and is the fifth president of ACI who in one way or another has had a hand in the early history of prepakt. He named the method.

WHERE PREPAKT IS USED

For the restoration of concrete structures which have suffered deterioration due to weather and other causes, the prepakt method has been employed for dams, tunnels, retaining walls, bridge piers and abutments, and reinforced concrete columns and beams in bridges and buildings. It has also been used for the stabilization of breakwaters and the encasement of piles. In a considerable number of cases old bridge piers resting on piles that have been undercut to such a degree that they were lacking in lateral stability have been repaired by jetting downward through drill holes in the pier or through inclined jet pipes beneath the pier to remove silt and clay. The cavities thus formed have then been filled by grouting. Sometimes such instability is further remedied by incasing the lower end of the pier in an extended footing of prepakt concrete.

Another type of old structure which frequently requires repairs is the stone masonry pier with shell of cut stone laid up in lime mortar and core of loose rocks or rubble masonry. Under the action of time and increasingly heavy loads such piers have become unstable. The piers have been repaired by raking the joints and by jetting through drill holes in the pier to remove disintegrated mortar, clay, silt, and fine sand, and then by pressure grouting through the drill holes and into the joints of the cut stone face so as to fill the joints and the voids of the core material.

Defective concrete surfaces are repaired by chipping until sound concrete is exposed; then the surface is customarily sandblasted to remove any partially fractured concrete caused by the chipping operation. After the cavity has been formed and completely filled with coarse aggregate, either in a moist condition or completely inundated for underwater work, it is grouted, always starting at the lowest point so that no grout will be required to flow downward but always upward and outward. Grouting is continued until undiluted grout appears at the outlet which is located at the uppermost point of the cavity. Where it is possible to do so, as when a cavity is completely enclosed, after closing the outlet as much pressure as the forms will safely hold is then applied to the grout and held for a short time; this practice insures optimum bond between old concrete and new prepakt.

While I have made no extensive inspection of the repairs to structures that have been restored by the prepakt method, from such information as has come to me, such repairs have generally given a good account of themselves with respect to strength, resistance to action of weather, bond between prepakt and old concrete, and absence of cracking and checking such as is frequently manifest in repairs made by more conventional methods.

Some notable examples of repairs made by the prepakt method are the filling of a large cavity caused by cavitation in the Arizona spillway tunnel of Hoover Dam and the rehabilitation of Barker Dam, increasing its stability and watertightness by a prepakt addition to the upstream face which had suffered severe deterioration. In both these cases the grouting was done through vertical pipes within the aggregate mass. For Barker Dam, which is about 175 ft high and about 700 ft long, as the reservoir was filled with the spring runoff from melting snows the coarse aggregate was precooled to a temperature of about 40 F before grouting through a maximum water depth of 175 ft was begun. Grouting was carried out as a continuous operation, thus producing 12,500 cu yd of prepakt concrete during the 10-day grouting period. By this precooling operation the average maximum temperature in the prepakt addition during the period which followed grouting, as the cement hydrated, was only 63 F.

A good example of stabilizing and strengthening old stone masonry piers substantially to increase load carrying capacity was that of the piers of Victoria Bridge on the Canadian National Railway crossing the St. Lawrence River at Montreal.

COMPOSITION OF PREPAKT

The coarse aggregate may be of any suitable maximum size, depending on the thickness or size of the mass. For thick sections and large masses it is desirable that nearly all of the coarse aggregate be retained on the ⅝-in. sieve and for structures of thin section, such as walls, it is desirable that practically all of the coarse aggregate be retained on the ½-in. sieve and only a small percentage pass the ⅜-in. sieve. While for conventional concrete, flat and elongated coarse aggregate particles are objectionable because of their effect upon water requirement and workability, for prepakt concrete particle shape of coarse aggregate is not a matter of importance in so far as the groutability of the aggregate mass is concerned. In the interest of economy of the more expensive materials used in the grout, it is desirable that the coarse aggregate be so graded and be of such particle shape and surface texture that after placement in the forms the void content of the aggregate mass will be a minimum. In practice the void content usually ranges between 33 and 45 percent, being least for a well graded, well compacted gravel of smooth surface texture and large maximum size, and greatest for poorly graded crushed stone of small maximum size.

The grout which is used in prepakt work is composed of portland cement; a pozzolanic material of low mixing water requirement (usually a fly ash of especially low carbon content and high fineness); sand, practically all of which passes the No. 16 sieve and a substantial percentage which passes the No. 100 sieve; a small amount of intrusion aid (usually about 1 percent of the weight of the cement plus pozzolan); and sufficient mixing water to produce a fresh grout of consistency suitable for the work.

As for conventional concrete, the preferred type of cement will depend upon the nature of the work to be performed; for most work Type I or Type II is used. The purpose of the pozzolan is to improve pumpability and penetrability of the grout; to retard the rate of stiffening; in thick sections, to reduce the heat of hardening; in hydraulic structures and tunnel linings, to decrease permeability; and to increase resistance to the action of sulfate and weakly acid waters.

The intrusion aid is a mixture of chemical agents which reduces the mixing water requirement, retards the rate of stiffening and setting of the fresh grout, improves pumpability and groutability, assists in keeping the solids in suspension, reduces bleeding, and produces an

expansion (because of the generation of tiny gas bubbles) prior to the time of setting.

The proportions of grouting materials commonly employed for prepakt construction as with conventional concrete, depend upon the desired strength. Within my knowledge, these proportions have ranged from as rich as 9 parts of cement to 1 of pozzolan to 6 of sand, to as lean as 1 part cement to 1.6 pozzolan to 5 of sand, measured by weight. For the rich mix and the particular coarse aggregate there employed, the cement factor was about 7¾ bags per cu yd; and the 1-year compressive strength was about 12,000 psi. For the lean mix the cement factor was about 1.7 bags per cu yd, and the 1-year compressive strength was in the order of 2500 psi.

As for conventional concrete, the ultimate compressive strength of prepakt is related to the ratio of the quantity of mixing water to the quantity of cementing materials. For pozzolans of the type and amount that are usually employed in prepakt grouts, the 90-day compressive strength of standard cured prepakt concrete is about the equivalent of the 28-day strength of corresponding conventional concrete for which the cementing medium is normal portland cement.

MIX PROPORTIONING

The problem of mix proportioning for prepakt is somewhat different from that of conventional concrete. The particles of coarse aggregate being in contact with one another, a cubic yard of mixed coarse aggregate is required for every cubic yard of prepakt concrete. Where several sizes of coarse aggregate may be employed, proportions of the several sizes to produce a coarse aggregate mass of minimum void content after placement in the forms may be determined from the results of several trials using blends in which the proportions of the several sizes are varied. The void content of any combination of sizes may be conveniently determined by placing the sample of mixed aggregate in a tank of appropriate size and then measuring the quantity of water which is required to fill the voids.

The desired proportions of the materials for prepakt grouts are likewise determined by trial mix tests in which the variables may be the type and quantity of cement, the source and quantity of pozzolan, the quantity and grading of sand, and the quantity of intrusion aid. The optimum proportions will be those that, for the desired consistency, will produce a grout of satisfactorily low water requirement for which the solid constituents will remain in suspension, the water retentivity will be high, the bleeding will be low, and the desired expansion prior to setting will be obtained.

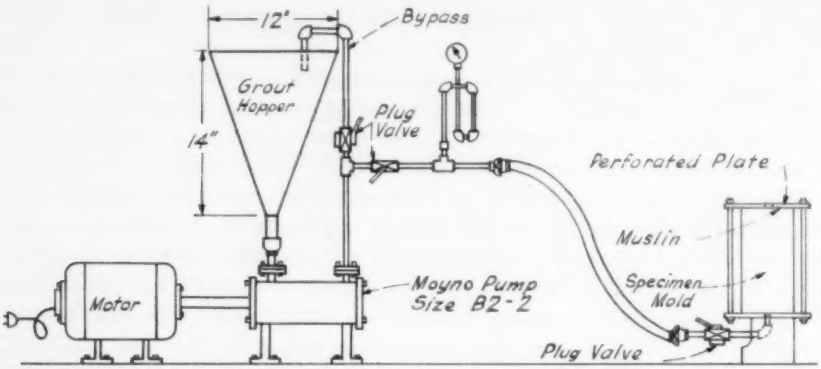

Fig. 2—Grouting apparatus for 6 x 12-in. prepakt concrete test cylinders

The compressive strength of a grout is not a satisfactory index of the compressive strength of prepakt in which the grout may be employed. For this reason the strength producing qualities of a grout are determined by measuring the compressive strength of test cylinders of prepakt concrete, in which the grout is employed.

For much prepakt repair work, it is important that the grout retain its mobility or fluidity for several hours. Factors which influence the rate of stiffening are the type or brand of cement, the quality of the pozzolan, the richness of mix, and the amount of chemical admixture or intrusion aid.

TESTING METHODS FOR PREPAKT

It is the general practice to determine the compressive strength of prepakt by tests on 6 x 12-in. cylinders with maximum size of aggregate 1½ in., the test cylinders being cured in the standard manner. Similarly, tests of other properties of hardened prepakt are made on the same types of specimen as those employed in investigating the properties of conventional concrete. The cylinders for compression tests are manufactured by first filling the 6 x 12-in. mold with coarse aggregate; then with end plates positioned as shown in Fig. 2, the aggregate mass is slowly grouted from the bottom by pumping until undiluted grout appears at the top outlet. Thereafter, the inlet and the outlet to the mold are closed and the pump is disconnected. If the coarse aggregate in the work is of maximum size greater than 1½ in., as will often be the case for thick sections, only the material of sizes less than 1½ in. is employed for filling the cylinder molds. This corresponds to the practice followed in the preparation of 6 x 12-in. test cylinders of conventional concrete where aggregate sizes greater than 1½ in. are removed by wet screening.

Specifications may set up grading limits for each of the coarse aggregate sizes, as for conventional concrete. Also specifications will always set up grading limits and uniformity requirements for sand. These will, of course, make necessary analyses to determine whether the aggregate materials being supplied meet with the specification requirements.

Two devices are used for measuring the consistency of grout. The first of these, called the "flow cone method," is always used in the field, much as would be the slump test for conventional concrete, as a means of obtaining the desired degree of consistency control. It consists in determining the time of efflux of a given volume (1725 ml) of grout from the flow cone apparatus shown in Fig. 3. The "flow factor" in seconds is the elapsed time of discharge, as determined by stop watch, after quickly removing the finger from the end of the discharge tube. The specifications may place upper and lower limits on the flow factor just as specifications usually place upper and lower limits on the slump of conventional concrete; for example, it might be required that the flow factor be no less than 15, nor more than 25 sec.

The second method for measuring consistency requires the use of the apparatus shown in Fig. 4, which is known as the "consistency meter." This more sensitive but less rapid method is used only in the laboratory. The angle of twist caused by shearing forces between the paddles or arms of the spider and the grout in the rotating pan is called the "consistency factor." A consistency factor of 180 deg is

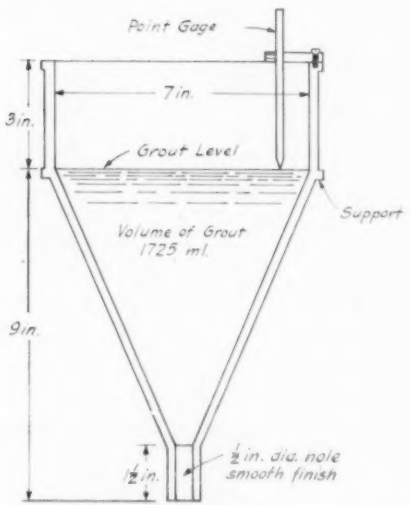

Fig. 3—Flow cone

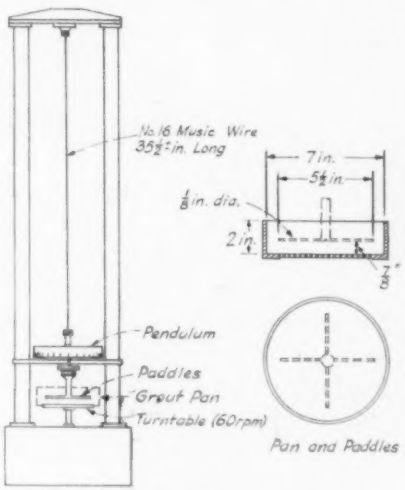

Fig. 4—Consistency meter

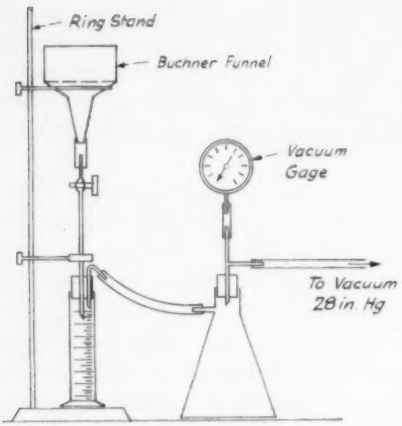

Fig. 5—Water extraction apparatus

roughly the equivalent of a flow factor of 20 sec. The consistency meter is considerably more sensitive to small changes in consistency, such as will be caused by small variations in water content, than is the flow cone; but the flow cone is quite as satisfactory and reliable for determining the consistency of fresh grouts as is the slump cone for determining the consistency of fresh conventional concretes.

The water retentivity of fresh grouts is considered to be a valuable index to quality. The apparatus for this test is shown in Fig. 5. The 500-mi Buchner funnel is lined with a sheet of 11 cm No. 42 Whatman filter paper. The 250-ml graduated cylinder is cut down to a volume of 130 ml. The vacuum gage is calibrated in inches of mercury and the vacuum system is capable of drawing a constant vacuum of 28 in. of mercury when evacuating a system having a volume of 1 liter. After the filter paper has been properly wetted and the excess water has been drawn off by vacuum, the sample of fresh grout is poured into the funnel until it is flush with the top. The vacuum of 28 in. of mercury is then applied to the funnel and the time required to remove 60 ml of water from the sample of prepakt grout is determined. This is designated as the "time of extraction," which is a measure of water retentivity. Specifications may place a lower limit on the time of extraction. For most prepakt grouts of normal richness of mix, a time of extraction of 5 min may be regarded as satisfactory.

Specifications frequently place limits on the permissible bleeding of fresh grouts up to the age of 4 hr and also upon expansion. For these tests approximately 900 ml of freshly mixed grout is placed in a 1000-ml glass graduate at a temperature of about 70 F. During this period the expansion of the sample, as indicated by the difference between the original level of the top of the grout column and the top of the bleeding water, may be periodically observed. It is generally considered that a total expansion of 50 to 100 ml during the 4-hr period is desirable. At the end of 4 hr the bleeding water collected on the surface of the grout is measured. As a general rule, for grouts of normal richness of mix, the quantity of bleeding water should not exceed 10 ml.

PROPERTIES OF PREPAKT

The heat of hardening, strength, drying shrinkage, permeability, resistance to weathering, and resistance to aggressive waters are affected by the quantity, fineness, and composition of the cementing materials (cement plus pozzolan), the charter and amount of the coarse and fine aggregates, the quantity of mixing water, and the quantity and distribution of entrained air or other gas. Considering the pozzolan as a part of the cementing material, when the total weight of portland cement plus pozzolan is equal to the weight of straight portland cement in corresponding conventional concrete, then quite generally the strength of prepakt will be less at the earlier ages than that of the conventional concrete; but under favorable conditions of curing, the strength of prepakt may be substantially greater than that of the straight portland cement concrete at the later ages.

Two properties in which prepakt excels are drying shrinkage and bond to old concrete. Its low drying shrinkage, which is likely to be only about 1/6 as great as the usual mortar applied pneumatically and less than half as great as that of conventional concrete, may perhaps be explained by the fact that prepakt contains a much larger percentage of coarse aggregate (1 cu yd of coarse aggregate for 1 cu yd of prepakt) and that the pieces of coarse aggregate are in point to point contact and not separated by a layer of cement paste, the principal contributor to drying shrinkage.

The much higher bond strength that is developed between prepakt concrete and old conventional concrete, compared to the bond strength that is ordinarily obtained between new and old conventional concrete, is probably due to the grout being applied under pressure. It possesses wetting properties and its fluidity is much greater than that of the usual mortar constituent of conventional concrete. When the surface of the old concrete has been properly prepared by sandblasting, numerous tests indicate that the bond strength developed between the old concrete and prepakt, as determined by modulus of rupture, approaches the flexural strength of the new prepakt or the old conventional concrete, whichever is the weaker in the beam element.

The bar diagrams on the right of Fig. 6 show the shrinkage of 3 x 3 x 40-in. bars caused by 60 days of exposure in an atmosphere of 50 percent relative humidity and 70 F. The same total quantity of cementing materials, sand, and water were employed for the prepakt as for the conventional concrete, though for the conventional concrete, the sand was much coarser, being a typical concrete sand. The conventional concrete contained straight portland cement; for the prepakt the cementing materials were composed of 2 parts of portland cement to 1 of a fine, low carbon fly ash. The specimens were standard moist cured

for 28 days before the beginning of tests. It will be noted that the drying shrinkage of prepakt was about one-half that of the conventional concrete.

In connection with the repair of the Arizona spillway tunnel of Hoover Dam, to which reference was earlier made, drying shrinkage tests were made on cores of prepakt taken from the work and also on cores taken from the conventional concrete of the original tunnel lining. After water soaking and then drying at 50 percent relative humidity and 70 F for 3 years, it was found that the drying shrinkage of the prepakt cores was only about ⅓ as great as that of the conventional concrete cores.

In an attempt to determine the relative crack resistance of prepakt and conventional concrete, 3 x 3 x 40-in. bars, in each of which was embedded axially a ¾-in. diameter steel rod, after 28 days of moist curing, were subjected to the same drying conditions as those for the shrinkage specimens. To provide anchorage, the rods were threaded for a short distance at each end; and to prevent bond between concrete and steel, the rods were rubber covered for the remainder of their length. The steel rods with their threaded ends restrained the concrete bars against shrinkage; and the tendency toward shrinkage produced compressive deformations in the rods which were observed with a special extensometer. With this information, and knowing the modulus of elasticity of the steel, the average tensile stresses in the bars, at any time, could be computed. The bar diagram at the left of Fig. 6 shows the computed average tensile stresses after 60 days of drying. For the prepakt the average tensile stress was 190 psi and for the conventional concrete 315 psi. Shortly thereafter, the tensile strength of the conventional concrete was reached and transverse cracks developed in the bars at about midlength. No such cracks ever developed in the prepakt bars although observations were continued for 3 years.

In the United States, in conventional concretes where resistance to freezing and thawing is desired, it is now the universal practice to employ an air-entraining agent. In prepakt concrete resistance to such weathering action is provided through the use of aluminum powder which is one of the constituents of the grouting or intrusion aid. This intrusion aid is normally employed in the amount of about 1 percent of the weight of cement plus pozzolan. When used in proper amount and of proper grade, this aluminum powder reacts with the alkalies liberated by the cement in the earliest stages of hydration to produce tiny bubbles of hydrogen gas in the grout prior to the time of setting, but mostly after the grouting operation has been completed, which bubbles are of proper size and spacing in the grout mixture to insure superior resistance to freezing and thawing action. Also incorporated

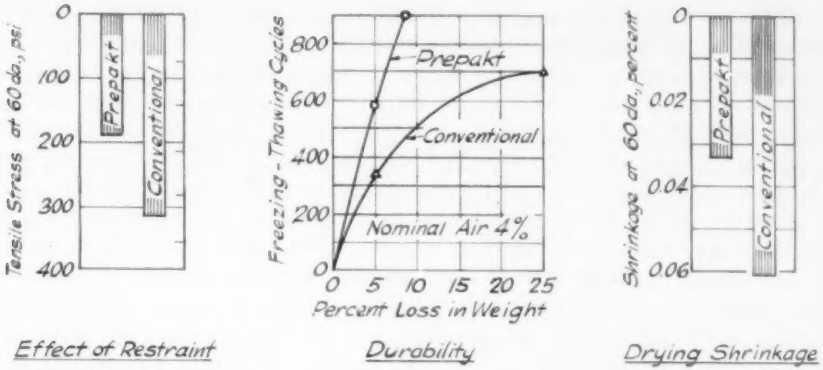

Fig. 6—Shrinkage and durability of prepakt concrete
Prepakt concrete: $W/(C+P) = 0.47$; 4000 psi at 28 days
Conventional concrete: $W/C = 0.48$; 5000 psi at 28 days

in the intrusion aid is a compound which will cause the aluminum powder to react before the setting time of the grout even when the portland cement may be of very low alkali content.

The middle diagram of Fig. 6 shows the relationship between loss in weight and number of freezing and thawing cycles of 3 x 6-in. cylinders frozen in water at −18 F and thawed in water at 50 F after standard moist curing for 28 days. At that age the compressive strength of the conventional concrete was about 5000 psi and that of the prepakt concrete was about 4000 psi. The nominal amount of entrained air or hydrogen gas was 4 percent. Resistance to freezing and thawing was judged by losses in weight, and the specimen was considered to have failed when its loss in weight reached 25 percent. For the treatment here described, a concrete is regarded as possessing satisfactory weathering resistance if it will withstand 300 cycles of freezing and thawing without failure.

From an examination of Fig. 6, it will be seen that this particular conventional concrete lost 25 percent in weight at 700 cycles; the prepakt concrete had only lost 8 percent of weight after 900 cycles, when the test was discontinued. Results shown in the figure should not be taken to mean that all prepakt concretes possess weathering resistance superior to air-entrained conventional concretes. There are, of course, many other variables that enter the picture; however, it may be fairly stated that, based on laboratory tests and on reported performance of field structures over a period of years, prepakt concretes of suitable materials and good workmanship may be expected to exhibit excellent resistance to weathering action due to freezing and thawing.

Characteristically, concretes containing pozzolanic materials, though generally exhibiting greater permeability at the very early ages, become substantially more watertight than do corresponding concretes containing straight portland cement at the later ages. For this reason, since prepakt grouts normally contain a fairly high percentage of a pozzolan of low water requirement, such as some of the very low carbon fly ashes, under moist conditions prepakt concretes become extremely watertight at the later ages. As an illustration, at the age of 6 months permeability tests were made on concretes for which the ratio of water to cementing material was 0.70, each containing the same total quantity of cementing material; for straight portland cement concrete the permeability coefficient (cu ft per sec per sq ft of area per ft of head per ft of length) was 15×10^{-12}; and for prepakt concrete for which the cementing material was in the ratio of 1 part of portland cement to 1 part of fly ash, the permeability coefficient was 2×10^{-12}. What has just been said should not be taken to mean that conventional concretes containing the same cementing materials in corresponding amounts and of the same ratio of water to cementing materials (cement plus fly ash) would not exhibit an equal degree of impermeability at the later ages.

STRENGTH OF JOB CYLINDERS VERSUS CORES

With prepakt, perhaps more than with conventional concrete, there is the question of the relationship between the compressive strength of job cylinders manufactured as heretofore described and cured under standard moist conditions, and the compressive strength of prepakt in the actual structure as determined by tests on cores. At Barker Dam where the maximum size of aggregate in the addition to the upstream face of the structure was 4½ in. and where grouting was carried out underwater at depths as great as 175 ft, the average compressive strength of 5-in. diameter cores was about 25 percent less than the average compressive strength of 6 x 12-in. job cylinders containing aggregate with a maximum size of 1½ in. Considering the large ratio of aggregate size to specimen diameter for the cores, it seems fair to assume that the strength of the prepakt in place was greater than that indicated by the cores (3800 psi) and perhaps approached that of the job cylinders (4700 psi).

In a rather extensive series of tests made by the Corps of Engineers, the 90-day compressive strength of 10-in. diameter cores for which the maximum size of aggregate was 3 in. and for which the ratio of water to cementing material (cement plus fly ash) was 0.60, was 4100 psi as compared with 4370 psi for 6 x 12-in. cylinders for which the maximum size of coarse aggregate was 1½ in. Also the 90-day compressive strength of

6-in. diameter cores taken from work in which the maximum size of aggregate was 2 in. and for which the ratio of water to cementing material for the grout was 0.42, was 6710 psi as compared with 6890 psi for 6 x 12-in. cylinders.

Data make it appear that under proper conditions of construction and good workmanship, the compressive strength of 6 x 12-in. job cylinders may be taken as a fair index of quality of prepakt in the actual structure.

SUMMARY

1. Prepakt concrete is produced by first filling the forms with coarse aggregate and then filling the voids of the coarse aggregate mass by grouting under pressure.

2. Conceived independently by the late Louis S. Wertz and the writer, it was first employed in 1938 for strengthening and stabilizing the concrete lining of Muir tunnel on the Santa Fe Railroad near Pittsburg, Calif.

3. Oddly enough, four past presidents of the Institute in one way or another had a hand in the development of the method, and from the 1960 president came the name prepakt.

4. While it is a method that is adapted to new construction, both underwater and in the dry, it has come to be widely used in the restoration and repair of concrete and masonry structures.

5. The minimum size of coarse aggregate present in any substantial amount should not be less than ½ to ⅝ in. The coarse aggregate may be of any convenient maximum size and of any grading; for maximum economy of grouting materials it is obviously desirable that the coarse aggregate be so graded that the void centent of the aggregate mass after placement will be a minimum. It is particularly important that the coarse aggregate be clean and free from stone dust, silt, clay, or other deleterious coating materials.

6. Prepakt grouts are composed of portland cement, pozzolan of low water requirement such as low carbon fly ash, fine sand, and a small amount of a chemical admixture in powder form, usually referred to as intrusion aid, with sufficient water to produce a slurry of cream-like consistency which will hold the solid constituents of the grout in suspension.

7. The pozzolan is carefully selected as a material to improve the properties of fresh grouts and substantially to contribute to greater strength and lower permeability of prepakt concrete at the later ages.

8. Intrusion aid used in prepakt grout, usually in the amount of about 1 percent of the weight of cementing materials (cement plus pozzolan), contains compounds which reduce the mixing water requirement, im-

prove fluidity, retard the rate of early stiffening and the time of setting, help to maintain the solid constituents of the grout in suspension, and produce an expansion of the grout through the generation of tiny gas bubbles prior to time of setting.

9. The sand for prepakt grouts should substantially all pass the No. 16 sieve and a substantial percentage should pass the No. 100 sieve. A lower limit of fineness modulus of about 1.5 is suitable for grouts which are to be employed in prepakt work of thin section where the coarse aggregate will be in the smaller sizes, say principally within the range ⅜ to ¾ in.; for thick sections or mass construction where the coarse aggregate is of large maximum size and where there may be only a small percentage of material below ¾ in., the sand may contain more of the coarser sizes up to No. 16 and the fineness modulus may be as high as 2.2 though generally a top limit of 2.0 is more desirable.

10. Mix proportions are usually based on the results of trial mix tests. The object is to produce a grout of minimum water requirement that for a given consistency will not exhibit undue early stiffening, will be low bleeding and of satisfactory water retentivity, and will exhibit the desired expansion during the setting period. A composition possessing such properties is selected that, with the coarse aggregate available, will produce a prepakt concrete of the requird strength. Standardized methods of test to determine the properties of fresh grouts and for the manufacture and testing of prepakt compression specimens are employed.

11. In general, at the early ages the compressive strength of prepakt concrete is lower than that of corresponding conventional concrete containing straight portland cement. When the comparison is on the basis of total cementing materials (which include cement plus pozzolan for the prepakt concrete), under moist curing conditions or in structures of thick section which are not permitted to dry out, the later-age strength of prepakt concrete may be considerably greater than that of corresponding conventional concrete containing straight portland cement.

12. The drying shrinkage of prepake concrete is very low, usually not more than ½ and in some cases not more than ⅓ of that of conventional concretes. This low drying shrinkage is thought to be due to the point to point contact of the pieces of coarse aggregate. Because of this low drying shrinkage, the tensile stresses set up due to drying are low; and as a consequence the crack resistance factor of prepakt is likely to be substantially greater than that of the usual conventional concrete.

13. From the examination of a number of jobs where both conventional and prepakt concretes were employed in repairs, it seems evident that, in actual service under comparable conditions, the degree of

cracking of prepakt concrete is substantially less than that of conventional concrete and is much less than that exhibited by most repairs that have been made by plastering with sand-cement mortars.

14. With old concrete surfaces properly treated by sandblasting, the bond between new prepakt and old concrete is much greater usually than is the bond between old and new concrete, the bond strength frequently equaling the tensile strength of the prepakt or old concrete whichever is the weaker.

15. For equally high water-cement ratios, the later-age permeability of prepakt concrete, because of the nature and amount of the pozzolan generally used as a constituent of the grout, is likely to be much less than that of corresponding conventional concrete containing straight portland cement, although obviously the permeability should be no greater than of a corresponding concrete using the same pozzolan in like amount.

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REFERENCES

1. Keener, Kenneth B., "Erosion Causes Invert Break in Boulder Dam Spillway Tunnel," *Engineering News-Record,* Nov. 18, 1943, pp. 762-766.

2. "Substructure Repairs Under Difficult Underwater Conditions," *Roads and Bridges* (Toronto), Nov. 1943, pp. 36-40, 65-69.

3. Disney, C. P., "Rejuvenates 85-Year-Old Bridge Piers," *Railway Age,* June 1945, pp. 1049-1051.

4. "Dry Packing and Grouting of Substructure Salvages 90-Year-Old Montreal Bridge," *Engineering News Record,* Jan. 10, 1946, pp. 120-122.

5. Kelly, J. W., and Keatts, B. D., "Two Special Methods of Restoring and Strengthening Masonry Structures," ACI JOURNAL, V. 17, No. 4, Feb. 1946 (*Proceedings* V. 42), pp. 289-304.

6. Keatts, B. D., "The Maintenance and Reconstruction of Concrete Tunnel Linings with Treated Mortar and Special Concrete," ACI JOURNAL, V. 18, No. 7, Mar. 1947 (*Proceedings* V. 43), pp. 813-826.

7. "Leaning Grain Elevator Restored by Ingenious Method," *Railway Engineering and Maintenance,* May 1947, 4 p.

8. Sandberg, C. H., "Santa Fe Again Tames Canyon Diablo," *Railway Age,* Dec. 1947, 4 p.

9. "Dam Refaced with Precast Concrete," *Engineering News-Record,* Nov. 13, 1947, pp. 116-120.

10. Davis, Raymond E.; Jansen, E. Clinton; and Neelands, W. T., "Restoration of Barker Dam," ACI JOURNAL, V. 19, No. 8, Apr. 1948 (*Proceedings* V. 44), pp. 633-667.

11. Davis, Raymond E.; Jansen, E. Clinton; and Neelands, W. T., "Grouted Gravel Fill and Precast Slab Provide New Face for Barker Dam," *Civil Engineering*, V. 18, Feb. 1948, pp. 78-83, and p. 136.

12. Downs, L. V., "Floating Caisson Facilitates Repair of Grand Coulee Spillway Bucket," *Civil Engineering*, V. 20, Apr. 1950, pp. 255-259.

13. Minnotte, J. S., "Lock No. 5 Monongahela River Refaced by Grout-Intrusion Method," *Civil Engineering*, V. 22, Oct. 1952, pp. 872-875.

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From ACI Committee 201 Symposium on Restoration of Deteriorated Concrete

Title No. 57-9

Repair of Damaged Concrete with Epoxy Resins

By BAILEY TREMPER

The use of adhesives and binders containing epoxy resins by California Division of Highways in repairing concrete is described. Illustrations of their use in repair work are given. The discussion includes possible variations in formulation to secure wanted properties for specific uses, methods of application that are necessary to obtain strong and durable repairs, and a typical formulation for general use.

■ THE CALIFORNIA DIVISION OF HIGHWAYS has repaired damaged concrete with adhesives and binders containing epoxy resins as the essential component. The term adhesive is used to describe a formulation for bonding discrete portions of concrete. The term binder is used to describe a formulation used as a cementing agent to bind particles of aggregate into a mass that is originally plastic but later becomes rigid; epoxy resin is the binder in epoxy concrete just as portland cement paste is the binder in portland cement concrete. Actually the same epoxy formulation can be, and frequently is, used either as an adhesive or a binder.

The first description of epoxy resin appears to have been published by a Norwegian, Lindeman, in 1891. However, the manufacture of epoxy resins for commercial distribution is a development of the past decade. Epoxy resins are condensation products of epichlorohydrin and bisphenol. They contain oxirane groups and ether linkages. When activated with suitable curing agents, they form strong, chemically resistant structures having remarkable adhesive properties to concrete, wood, steel, and many other substances. Epoxy resins as presently manufactured, are available in a wide range of consistencies ranging from a virtual solid to liquids of relatively low viscosity. Those of higher viscosity have higher molecular weights. The color ranges from straw to amber.

At least four manufacturers in the United States now produce and market epoxy resins under license. As far as known, comparable grades of different producers have similar properties. The grades of epoxy

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resins are generally specified by viscosity and epoxide number or epoxy content. ASTM D 1652-59T gives methods of determining epoxy content, percentage of oxirane oxygen, and weight per epoxy equivalent of epoxy coating resins.

Epoxy adhesives were first used by the California Division of Highways in 1954. The discovery, originating in the laboratory of this division as far as known, that epoxy adhesives will form a strong, durable bond between fresh plastic concrete and old, hardened concrete, has greatly extended the usefulness of this compound as a repair material.

MATERIAL REQUIREMENTS

Only the resins of low viscosity find application as concrete repair materials. When cured, these resins are exceedingly strong but, without modification, possess little flexibility, particularly in cold weather. Polysulfide polymers can be added to improve flexibility without serious sacrifice in strength. The bond strength of a properly formulated adhesive is greater than that of concrete. This has been demonstrated many times by cementing two parts of a broken beam together and loading it to failure in flexure. Rupture always occurs in the concrete, not in the adhesive bond (Fig. 1). Similar tests in shear or in tension result in failure of the concrete itself. When fresh concrete is cemented to old concrete, failure occurs in whichever portion of the concrete is the weaker, not in the adhesive bond.

Adhesives and binders for highway use are formulated of epoxy resin, a plasticizer, and a curing agent. Depending on the viscosity at the time of mixing, it is usually possible to incorporate a certain amount of finely ground inert filler such as talc or silica. The amount of filler that can be used depends on ambient temperature and the viscosity desired for application. Primary components of low viscosity are desirable from the standpoint of reduction in cost by the maximum use of filler and ease of mixing.

Epoxies cure with an exothermic reaction. Large masses harden more rapidly than do small ones. The time required for the compound to harden sufficiently to withstand applied load, therefore, depends on its mass and temperature. Moderate heat can be applied to accelerate curing.

For the repair of highways and bridges under traffic, it is desirable to secure hardening as quickly as possible, but this complicates the

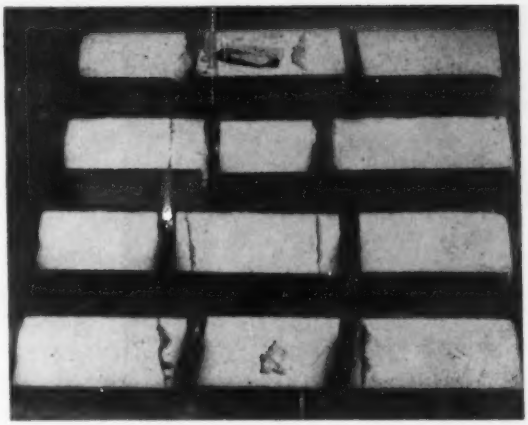

Fig. 1—Broken concrete cemented with epoxy adhesive and later rebroken

mixing and handling of the compound since rapid hardening formulations also have a short pot life, or period after mixing before hardening starts. In general, small batches must be mixed and applied promptly.

It is obvious that the final combination of the ingredients must be accomplished at the site of the work. Although three components are generally used, the plasticizer and curing agent can be packaged together if they are mutually soluble. Some primary compounds will function both as a plasticizer and curing agent. It is possible to incorporate some filler in the packages if the conditions of use are known in advance. In general, it is desirable to incorporate most of the filler at the time of mixing and thus secure better regulation of the consistency. Pigment grade tinting colors can be added to match the concrete if desired.

Curing agents are generally amines and are available from a number of manufacturers. The choice of curing agent and the percentage used determines the pot life and the time required to harden.

PATCHING TECHNIQUES

In patching spalled areas with epoxy mortar, or epoxy concrete, the binder used by the California Division of Highways contains little or no filler and has a pot life of about 20 min at 70 F. Under favorable conditions the repair has been opened to traffic within 3 to 5 hr. In cool weather metal channels are placed over the repaired area and they are warmed with torches (Fig. 2).

The aggregates used in epoxy mortar and concrete must be surface dry as otherwise the mixtures will not harden. Using a fine sand in mortars, the ratio of epoxy binder to sand can be of the order of 1 to 7 by weight. The economy in using coarse sand is not great. If the volume of the repair is large enough, both coarse and fine aggregate

Fig. 2—Placing epoxy concrete (foreground). Warming epoxy concrete (background). Repair opened to traffic in 2 hr

can be used and the ratio of epoxy binder to aggregate can be as large as 1:18 by weight. Mixing may be accomplished by hand or machine methods. The amount of binder is regulated to produce a plastic mass with sufficient excess binder to wet the surface to which the patch is applied. Alternatively the surface to be repaired may be primed with adhesive. The mixture is pressed or tamped into place and the exposed surface is brought to grade by screeding or troweling. The cost of epoxy concrete is of the order of $200 per cu yd. Nevertheless, it has been considered to be economical to use it in fairly large volume on heavily traveled structures such as the San Francisco-Oakland Bay Bridge.

If substantial volumes of repair work are involved and if traffic conditions permit, it is more economical to use portland cement concrete bound to the original work with a coating of epoxy adhesive (Fig. 3 and 4). It is believed to be essential in such work that the portland cement concrete have a low slump, not greater than 2 in. The concrete must be in place before the adhesive has started to harden as evidenced by the development of a tacky condition.

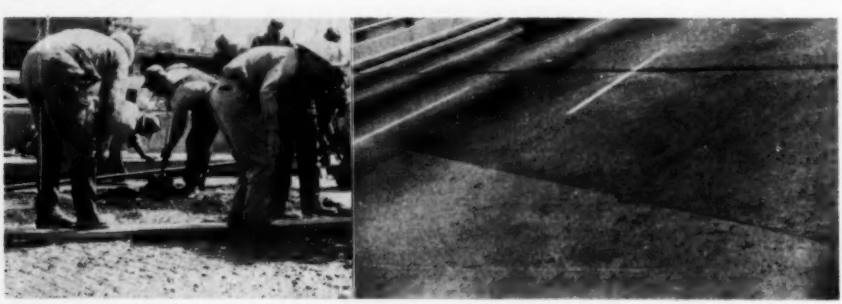

Fig. 3—(Left) Placing portland cement concrete on old concrete coated with epoxy adhesive. (Right) Repair after 15 months under traffic

Slower curing epoxy mixtures can be used as adhesives for fresh portland cement concrete since the time of hardening of the latter determines the time available for the adhesive to harden. Although curing agents are available to extend the pot life greatly, experience in the California Division of Highways has not been sufficient to conclude that they will produce an adequate degree of bond. New curing agents are being developed in considerable number and thorough testing and experimentation with them is warranted.

It is of utmost importance that the surface to which an epoxy adhesive is applied be scrupulously cleaned. Experience has shown attempts to remove oil stains from concrete with solvents have resulted in failure of bond. Sandblasting is the preferred method of cleaning. There are references to etching with dilute muriatic acid in distributors' literature, but the author cannot speak from experience in this matter. Concrete may be damp but must be free from surface moisture at the time the adhesive is applied.

Epoxy adhesives have been applied with squeegees, rollers, or inexpensive brushes which are discarded at frequent intervals. They are applied to a film thickness of about 40 mils (40 sq ft per gal.) with considerable excess on rough textured concrete.

Fig. 4a—Applying epoxy adhesive to bridge deck preparatory to placing portland cement concrete

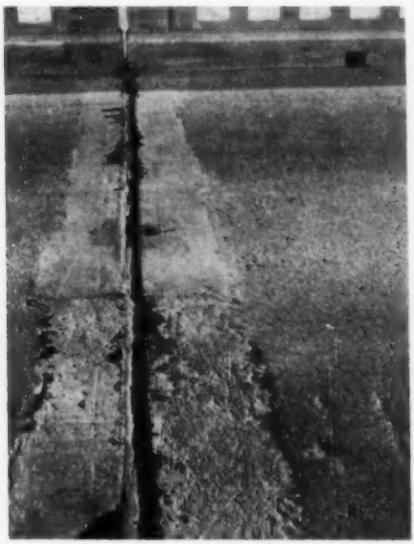

Fig. 4b—Typical condition of bridge deck before repair (foreground). Completed repair with portland cement concrete (background)

Fig. 5—Cracked bridge haunch repaired by injecting epoxy adhesive

The usual epoxy adhesive is too viscous for spray application but can be reduced to spraying consistency by the addition of aromatic solvents such as xylene. When a solvent is used it is important that the applied adhesive be allowed to stand until complete evaporation of the solvent has taken place. On the other hand, the waiting period must not be long enough to permit the development of set. Serious failures to secure bond to the new concrete have been observed when these precautions were not observed. Spray application using solvents is not recommended except under supervision by highly trained personnel.

Quick setting epoxy formulations should be mixed in small quantities, preferably not more than 2 gal. at a time. A dough mixer is convenient for this purpose. Disposable ice cream cartons are used in lieu of a metal mixing bowl to eliminate cleaning. A metal mixing paddle of appropriate shape is used.

Equipment used in mixing and applying adhesives and binders can be cleaned with aromatic solvents if done promptly. Once hardening has started, it is nearly impossible to remove the material except by strenuous mechanical means.

Cracks in concrete have been bonded (Fig. 5) by inserting small pressure fittings into drilled holes along the cracks at intervals of about 3 ft. The fitting is then cemented in place and the surface of the crack is sealed with adhesive leaving small vent holes. After this application has hardened, adhesive of low viscosity and containing little or no filler is injected through the fittings under considerable pressure.

A hand type grease gun or a pot operating under air pressure is used. A grease gun is preferred because of the higher pressure that can be developed. In addition to sealing cracks, this type of repair has been used to fill voids beneath metal expansion plates in bridge decks. Holes are first tapped in the metal plate and fittings are inserted. Surprisingly large quantities of adhesive have been injected into voids of this nature. If the plate is warmed with a torch, traffic can be permitted to pass over the repair within a few hours. The work has been eminently successful in eliminating looseness in bridge expansion plates.

The oldest work with epoxy adhesives performed by the California Division of Highways has been in cementing small reflective traffic markers to pavements. These have withstood heavy traffic at crosswalks for periods up to 5 years without failure in bond. This experience is cited as an illustration of proven durability of epoxy adhesives.

Epoxy installations in California have been made in regions of mild temperature except that recently a few have been made in locations where winter temperatures fall below zero. The author is not prepared to state at this time that the typical formulation yields adequate flexibility for exposure to cold weather.

The thermal compatibility of epoxy mortars and concretes with respect to the concrete to which they are applied needs more study particularly when relatively large areas are to be patched.

FORMULATIONS

The formulation most frequently used by the California Division of Highways both for an adhesive and a binder consists of:

Epoxy resin	10 parts by weight
Polysulfide polymer	4 parts by weight
Curing agent	1 part by weight
Filler	Variable

The epoxy resin has an epoxide number of 175 to 205 and a viscosity at 25 C of 5 to 9 poises.

The polysulfide polymer is a difunctional mercaptan having a specific gravity, 20/20 C of 1.27 and a viscosity at 25 C of 7 to 12 poises. Two curing agents have been used. One is 2,4,6-Tri (dimethylaminomethyl) phenol. The other curing agent is dimethylaminomethylphenol. The first agent yields a pot life of about about 20 min at a temperature of about 70 F. The pot life provided by the second agent is somewhat longer and is of the order of 45 to 60 min at 70 F. Filler is a finely ground, pigment-grade silica.

The epoxy resin is packaged separately. The polysulfide polymer and curing agent are mutually soluble and are packaged together.

A limited amount of construction involving epoxy adhesives has been performed under contract. One of the inspection problems has been in connection with securing thorough mixing of the two components of the adhesive. Recent specifications call for tinting the epoxy with titanium dioxide and the other component with carbon black. The mixed adhesive is required to be uniformly gray in color without visible streaks of black or white.

For some purposes the adhesive components have been made up with a predetermined amount of filler. In this case, finely divided talc has been used because of its antisettling properties. Colloidal silica in small amount has been added also to minimize sagging at the edges of horizontal applications or running in vertical applications. One manufacturer of reflective traffic buttons is packaging epoxy adhesive in a two part plastic bag. At the time of use, the separator is removed and mixing is accomplished by thorough kneading of the bag. A small opening is then made in the bag for dispensing purposes. The quantity of adhesive so packaged is about 1 lb which would be sufficient for small repair work.

Another type of epoxy combination is available commercially by a number of producers operating under license from the patent holder. It consists of two components, one of which contains a fluid epoxy resin; the other is "composed of a bitumen specially treated with an amine." Evidently the bitumen is largely coal tar. The mixed adhesive is rather low in viscosity and for some purposes filler can be added. Its pot life is relatively long, possibly 1 hr at 70 F. It requires from 2 to 10 hr to gain appreciable strength. Its main use to date has been as a thin resurfacing over slippery or otherwise deteriorated pavements and bridge decks. Sand or grit is applied to the surface to impart nonskid properties. Resurfacing of pavements and bridge decks with this type of composition is described in a Highway Research Board publication.[1] This publication is of particular interest because it describes a method of test for the strength or soundness of the surface on which it is proposed to construct the resurfacing. The test can also be used to evaluate the adequacy of preparation of the surface with respect to cleanness. According to literature circulated by one manufacturer of epoxy-bitumen (epoxy-coal tar) materials, it was placed at 17 locations on pavement, bridges, and airports involving a treated area of 37,650 sq ft during 1958. Epoxy-asphalt formulations are also being investigated.[2]

The cost of the liquid components of light colored epoxy adhesives or binders is about $9 per gal. When furnished and packaged by commercial formulators, the selling price may be of the order of $18 to $25 per gal. The cost of epoxy-bitumen systems, which are black, may be considerably less.

Epoxy-coal tar compounds have excellent adhesion to steel and hardened concrete. Bond to fresh concrete is relatively poor as determined by laboratory tests in the California Division of Highways.

Epoxy resins as such, or after modification with plasticizers and curing agents, can present a potential dermatitis problem. The effect is most severe with the more fluid types of epoxy resins. Workmen should protect the hands with protective cream and rubber or disposable plastic gloves. They should wear coveralls or other suitable clothing. All skin areas contaminated by resins or uncured mixtures should be cleaned immediately. Individuals after suffering an attack of dermatitis may become unduly sensitized and smaller quantities of these materials may thereafter cause distress and make it more difficult to avoid the condition.

The use of epoxy compositions for repairing damaged or deteriorated concrete is receiving wide attention. Informal discussions at scheduled sessions have been held at the 38th and 39th annual meetings of the Highway Research Board. An ad hoc subcommittee of ASTM Committee C-9 has recommended the formation of a permanent subcommittee on epoxy formulations for concrete construction. The U. S. Air Force has issued a pamphlet[3] on repairs to airfield rigid pavement using epoxy systems, to which are attached purchase descriptions for epoxy adhesive and epoxy binder for mortars and concretes.

Epoxy adhesives and binders should not be confused with other compounds such at polyvinyl acetates, polyesters, neoprene emulsions, and styrene-butadiene copolymer dispersions in water. Such compounds can function as adhesives, binders, or modifiers of portland cement paste and under certain conditions can develop strengths that are adequate for the work in hand. None, however, appears to possess fully the useful characteristics of properly formulated epoxy systems which include the ability to establish strong and permanent bonds to both fresh and hardened concrete, the ability to cure and harden in contact with moist concrete, negligible volume change on curing, the retention of cured strength in the presence of water, and outstanding resistance to weather and chemical agents.

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REFERENCES

1. Nagin, H. S.; Nock, T. G.; and Wittenwyler, C. V., "Development of New Nonskid Road Surface Treatment," *Bulletin* No. 184, Highway Research Board, Jan. 1957, pp. 1-9.

2. Miles, T. K., "Laboratory Work and Field Installations of Epoxy-Asphalt Concrete Pavements," Presented at 39th Annual Meeting, Highway Research Board, Jan. 11-15, 1960.

3. "Repairs to Airfield Rigid Pavement," *Air Force Pamphlet* AFP 88-116-1, Aug. 28, 1959.

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Title No. 57-10

From ACI Committee 201 Symposium on
Restoration of Deteriorated Concrete

Pneumatically Applied Mortar for Restoring Concrete Structures

By O. N. KULBERG

Pneumatically applied mortar is the most economical and successful means of restoring concrete structures where deterioration is relatively shallow and the restoration areas large and irregular. However, periodic protective applications are necessary in areas of severe exposure to seal hairline shrinkage and temperature cracks that may pass water and perpetuate deterioration of the parent structure.

Large deep deteriorated areas under severe exposure are economically restored by the installation of a metallic membrane.

■ The Southern California Edison Co. has used pneumatically applied mortar for the restoration of concrete structures over a period of 40 years with generally good success. In no instances has this mortar shown direct deterioration from weathering such as the scaling and spalling shown by low-air-entrained conventional concrete under severe exposure.

In addition to the large area of pneumatically applied mortar used in concrete repair, the company has several million square feet of this material in canals, flow lines, and on various structures throughout the system that is practically as good today as when it was applied. However, there are areas of severe weathering in the Sierra Nevada Mountains where pneumatically applied mortar restoration work has not been as successful as hoped for principally because of failure of the parent material upon which it had been placed or for lack of understanding on how to maintain it. In general, all pneumatically applied mortar regardless of how maintained has had sufficient longevity to justify its use. Since the less successful applications present the real experience challenge, this paper is devoted to these marginal problems and their solution.

HUNTINGTON LAKE DAMS

Pneumatically placed mortar was first used by Southern California Edison Co. in 1917 on the Huntington Lake dams to seal a construction joint between an initial development elevation of the Huntington Lake reservoir and a final elevation at a later date. The three dams which

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form the Huntington Lake reservoir, one of the company's hydroelectric storage reservoirs, are located in the Sierra Neveda mountain range in California approximately 40 miles airline southeast of Yosemite National Park and at an elevation of 7000 ft. These dams are a combination arch and gravity design and the initial development was built in the fall and winter of 1912, the concrete using local gravel deposits in the reservoir area, and the concrete was considered good by standards of that time. The final raising of the dams was started in the fall of 1916 and after a 3 month winter shutdown, due to unfavorable weather, was completed in the summer and fall of 1917. Quarried granite aggregate was used as the aggregate pits used on the initial development were inundated in the reservoir area. An unreinforced band of pneumatically placed mortar 20 ft wide and approximately 1 in. thick was placed over the construction joint on the upstream face of these dams between the initial elevation and the final elevation and large areas of this pneumatically placed mortar can still be seen on the remaining face of one of the three dams that has not been either completely backfilled or covered by metal facing (Fig. 1).

This particular reservoir stores spring runoff, the bulk of which occurs during the months of May, June, and July and is released for power generation during the fall, winter, and early spring months. This kind of an operation exposes the dams to a widely fluctuating water surface that is frozen most of the winter months, and the areas not covered by lake water are exposed to many cycles of freezing and thawing throughout the winter from snow that melts during the day and freezes during the night. After 20 years of exposure to this almost completely unsaturated (10 parts per million of dissolved solids) and mildly acid (pH factor 6.2) snow water, surface disintegration of both the 1912 and 1916 concrete reached a point requiring remedial treatment.

The surfaces of these dams showing deterioration were the surfaces exposed to the fluctuating lake water and the top and downstream areas which were exposed to the elements. Areas below the fluctuating water surface and areas under minor backfill retained their original soundness. This prompted backfilling the downstream faces of all dams with soil blanketed with rock to prevent erosion and placing several inches of bitumastic road mix surfacing on the top surface of the dams. Since the original 1918 pneumatically applied mortar sealer band material showed no signs of disintegration similar to that exhibited by the

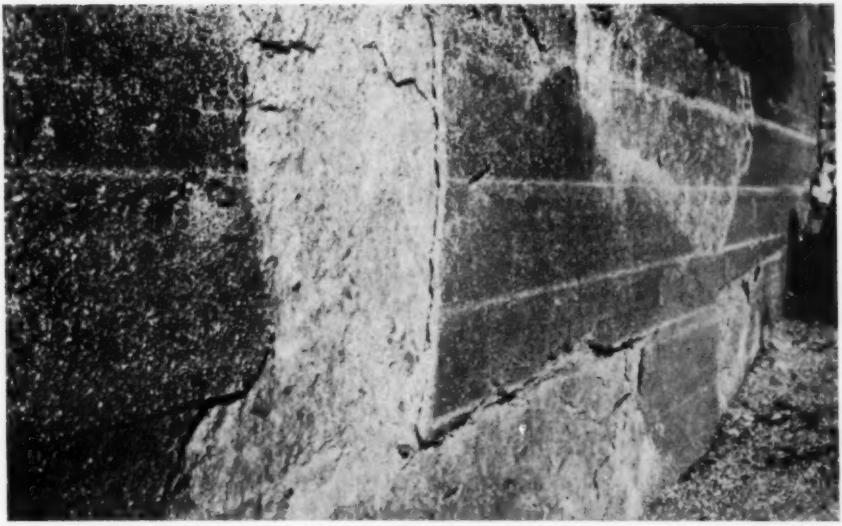

Fig. 1—1917 pneumatically placed mortar on upstream face of Huntington Lake Dam No. 3

concrete, several designs of pneumatically placed mortar were again tried on the upstream areas having the greatest need for remedial treatment in an effort to determine the most effective and economical restoration treatment to use.

On areas showing the greatest deterioration, all disintegrating material was removed with pneumatically operated multipoint bits, the margins keyed into sound concrete, and a 2-in. mesh of No. 9 welded wire fabric was dowelled to the face of the dams. It was covered with a minimum of 2 in. of 1:4 pneumatic mortar made from granite sand recovered from a tunnel waste pile. In other areas not as badly disintegrated, only the disintegrated surface material that could be removed with prospector picks was removed, the margins also keyed into sound concrete, and the area covered with a minimum of 2 in. of the same mortar as was used on the reinforced areas. Other disintegrated areas were spot chipped with pneumatic tools and the areas so cleaned were refilled with pneumatic mortar. Periodic observations of these areas revealed fine hairline shrinkage cracks in the larger areas having marginal restraint and after a few years service some of the cracks in the unreinforced areas developed small back flows as the reservoir receded. This condition gradually extended into all areas. In areas where the mortar was not reinforced and dowelled to the concrete, much of the coating pulled away from the concrete taking a thin layer of concrete with it (Fig. 2).

This treatment deferred further maintenance on these structures for 16 years from 1939 to 1955 until core tests and sonic strength determinations revealed deterioration of the concrete under the mortar and showed the necessity for a watertight membrane that would completely shut out all lake water. Since earth blanketing proved to be the most economical protection for concrete in the area, backfilling was again resorted to on the upstream faces of all dam areas that could be backfilled without interfering with the normal operating functions of the dams, such as sluice ways and water diversion facilities. Where it was impracticable to use an earth blanket, a 10-gage welded steel membrane facing was installed and keyed well into sound concrete under backfill or sound concrete below the minimum fluctuating water level of the reservoir.

The decision to install an earth blanket and steel membrane facing to check surface deterioration was purely an economic one. One arch of the Florence Lake multiple arch dam had been covered with a 12-gage black iron facing in 1942 as a test for this kind of facing, and in spite of many pessimistic predictions about rusting from the back and ability to effectively hold welds at the seams, the test was surprisingly successful (Fig. 3). This membrane has now been in service 18 years. Exami-

Fig. 2—Upstream face of Huntington Lake Dam No. 1 just prior to covering with steel membrane

Fig. 3—Arch of Florence Lake multiple arch dam waterproofed with a steel membrane

Fig. 4—Upstream face of Huntington Lake Dam No. 1 showing start of installation of steel membrane

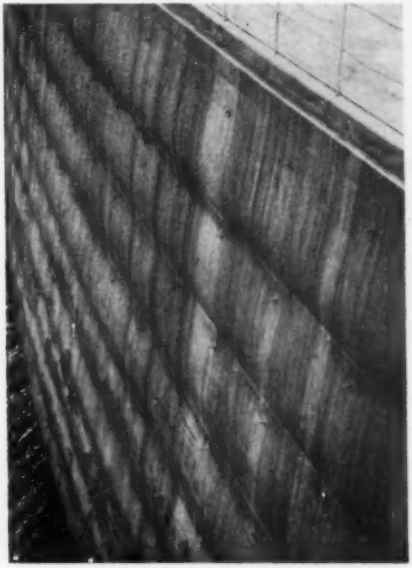

Fig. 5—Steel membrane on upstream face of Huntington Lake Dam No. 1 after 4 years of service

nation coupons have periodically been cut from it and up to the present, rusting on the unpainted back surface is surprisingly minor and of no material consequence.

Remedial work on the Huntington Lake dams had to be done when the reservoir could be drawn at least 10 ft below the normal minimum operating level in order that the steel facing could be keyed into sound concrete well below the frost line (Fig. 4). This could only be done during the most severe part of the winter when the reservoir inflow was a minimum and all work had to be completed ahead of the rising water of the reservoir as it filled, which normally required about 60 days. Studies showed that a steel membrane anchored out about 6 in. from the deteriorated dam face would have a twofold purpose, serving as a form to retain backup concrete as well as a waterproofing membrane and could be successfully applied within the time limits available and could withstand the weather handicap. To get the work completed in the available time, preparatory work such as scaling only the very loose material, setting anchor bolts, and cutting the margin key to which the steel membrane had to be attached had to be completed as the reservoir receded. This left only the placing and welding of the steel membrane, sealing the marginal keys with pneumatically applied mortar, placing the backup concrete, and painting to be completed as the reservoir filled.

A drainage system was provided at the junction of the new and old concrete to remove any water that might get behind the new facing. This drainage system consisted of 4 in. vertical gutter sections spaced 20 ft on centers and secured to the face of the old concrete. These vertical drains were connected to an 8-in. collector gutter along the bottom margin of the new facing and this in turn drained into an 8-in. core hole drilled through the dam at the lowest point of the new facing for gravity drainage.

Some seam repairs were necessary during the first season's operation. However, that leakage was surprisingly small and maintenance now consists only in occasionally spot painting areas scuffed by floating objects in the lake. This facing has now been in service for 4 years (Fig. 5).

FLORENCE LAKE DAM

The entire walkway on top of the Florence Lake multiple arch dam, with the exception of ten arches used as a long time test area by the Portland Cement Association, was restored by pneumatically placed mortar about 20 years ago and to date this mortar is as sound as when applied (Fig. 6). However, it does receive periodic coatings of hot linseed oil to seal fine shrinkage cracks that develop. This dam is in the same geographical area and at approximately the same elevation as the Huntington Lake dams and is subjected to the same exposure except that this reservoir is completely emptied each winter.

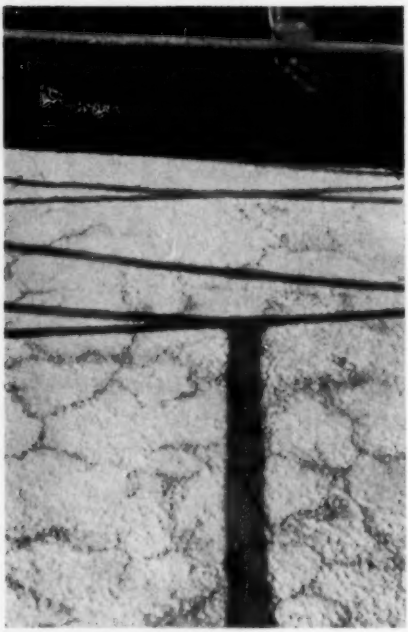

Fig. 6—Shrinkage cracks on walkway on Florence Lake multiple arch dam

The Florence Lake multiple arch dam is made up of 58 arches supported by counterforted buttresses spaced 50 ft on centers and was built during the summer and fall of 1925 and 1926. After 15 years of service, dampness and deposits of calcium carbonate on the downstream faces of the arches revealed the need for a waterproof membrane.

Pneumatically placed mortar was applied to two of the arches, one short one and one long one, for observation of its waterproofing ability. A third arch, previously mentioned, was covered with a 12-gage black iron membrane and was also used for observation. The two arch-

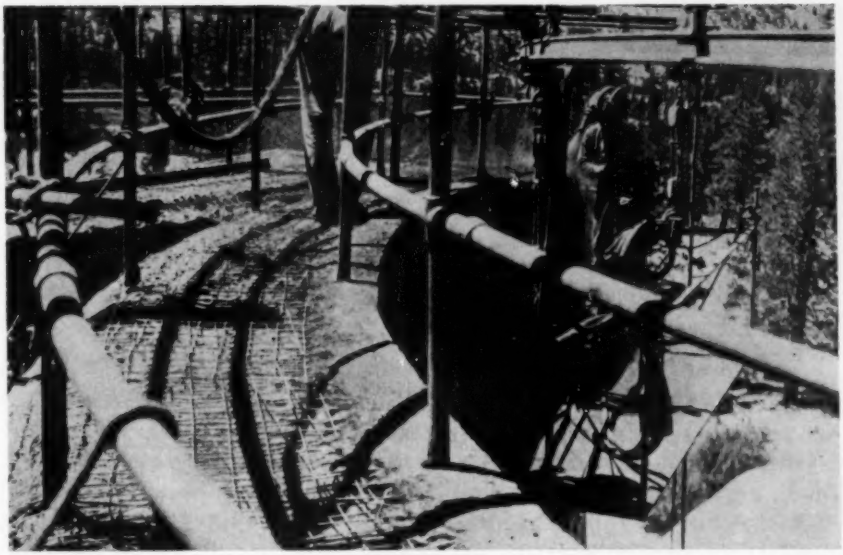

Fig. 7—Applying pneumatic mortar to the walkway of Florence Lake multiple arch dam

es coated with pneumatically placed mortar were scaled to sound concrete and coated with a 2-in. layer of reinforced pneumatically placed mortar which was made of 1:4 mix of selected well graded local river sand reinforced with a 2-in. mesh of No. 9 welded wire fabric dowelled to the face of the arches. The short arch, which had a sloping face approximately 40 ft long, was surprisingly tight and kept the downstream face completely dry. However, the long arch, which had a sloping face of approximately 140 ft continued to show damp areas at the locations of the daily construction joints. The slight movement which occurs at the construction joints during the winter when the reservoir is empty and the temperature varies from below zero to as high as 80 or 90 F could not be restrained by pneumatically applied mortar and consequently dampness and leaching continued through the long arch. The arch covered with the 12-gage metal face had a sloping length of approximately 100 ft and it completely sealed all dampness from reaching the downstream face and still does as long as the margin seals are maintained.

Shortage of steel during the war years precluded any further waterproofing with sheet steel. However, since it was absolutely essential that the penetration of moisture through the relatively thin arch sections of this dam be checked, a laminated felt and asbestos sheeting impregnated with tar (Asbestile) was applied to all arches except the short pneumatically placed mortar coated arch and the arch covered with the metal membrane and all have effectively sealed out lake water. The Asbestile

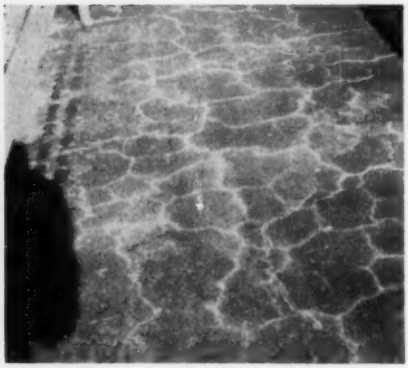

Fig. 8—Walkway on Shaver Dam repaired with conventional concrete

coated arches require annual maintenance in the spring to repair facing damaged by floating objects or movement of attached snow. This maintenance is relatively inexpensive as the annual damage is surprisingly small.

Disintegration of the corner edges of the walkway and corners of the sloping downstream buttress sections adjacent to and just under the walkway that are exposed to dripping snow water also required remedial treatment as disintegration had penetrated into the first grid of reinforcing steel. All such areas were chipped to sound concrete and covered with a layer of 2-in. mesh No. 9 welded wire fabric (Fig. 7) dowelled to the concrete and built back to the original alignment with a 1:4 pneumatically placed mortar made from a well graded natural river sand. Much of this pneumatically placed mortar restoration is now more than 20 years old and is still in an exceptionally sound condition. Practically all pneumatically placed mortar in this area develops fine hairline shrinkage cracks and they must be sealed if leaching and frost action is to be avoided. As soon as cracks are noticed, the entire pneumatically placed mortar area is flooded with two coats of hot (150 deg) linseed oil and coated with a light repellent paint. This treatment is good for about 5 years and has proved to be relatively cheap maintenance. Wherever practical when applying pneumatic mortar, lead-off troughs are installed to get rid of the snow water and keep the concrete dry.

Before the advent of entrained air as a durability additive for prolonging the life of concrete, and such additives are now being used with remarkable success by the Edison Co., the company had many theories and test programs. Practically all test programs were setup to compare the durability of the chosen concrete design with a pneumatically applied mortar using similar aggregate and cement. One such test location was the walkway on top of Shaver Dam. After 12 years of weathering under exposure similar to the Huntington Lake and Florence Lake areas this walkway was beginning to show the same type of corner disintegration

Fig. 9—Walkway of Shaver Dam repaired with pneumatically applied mortar

as found at the Huntington Lake and Florence Lake areas. Shaver Dam is a concrete gravity dam constructed from quarried granite aggregate and has a walkway 4 ft wide on the top.

About 20 years ago disintegration on the corner edges of the walkway had progressed to a point where remedial treatment was required. The walkway was chipped to sound concrete and a 4-in. layer of "no-slump" accurately controlled concrete made from carefully graded natural aggregate, mechanically vibrated, hand trowelled, and properly cured, was placed on the entire length of the walkway with the exception of a 100 ft strip on one end. This strip was coated with the equivalent thickness of pneumatically placed mortar using a 1:4 mix of the same well graded natural sand and cement and was used to compare durability with the above treatment. Both areas are sound because they both have been maintained with hot linseed oil. However, the cracking on the concrete area (Fig. 8) generally seems a little more pronounced than on the pneumatically applied mortar areas (Fig. 9), and seems to indicate that the pneumatically placed mortar is standing up a little better than the conventional concrete.

SUMMARY

Edison Co. experience indicates that reinforced and dowelled pneumatically placed mortar when properly maintained is the most economical method of restoring disintegrating concrete structures where disintegration is relatively shallow and over large irregular areas. However, it must have proper maintenance when disintegration is caused by aggressive water and frost action. By the same reasoning any concrete structure can have indefinite life, assuming ingredients are compatible, provided it can be reached, its condition is understood, and proper maintenance is exercised. Until such time as there is a "warranted concrete" that will withstand aggressive attack and frost action, it is prudent judgment, in areas where disintegration has been observed, to periodically check concrete structures, by sonic inspection or any means that will

reveal incipient deterioration, while it is still possible to apply economic deterrents without more costly restoration work.

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Resistance to Shear of Reinforced Concrete Beams

Part 1—Beams without Web Reinforcement

By J. TAUB and A. M. NEVILLE

The redistribution of internal forces in a beam in which the diagonal tension crack has extended to the level of the main tension steel is discussed. It is shown that the resistance to shear of a simply supported beam without web reinforcement depends on the strength of the compression zone at the upper end of the diagonal tension crack and of the tension zone at its lower end. Accepted methods of calculation of shearing stresses give, therefore, an incorrect idea of the strength of the beam, and result in a variable factor of safety.

Different types of shear failure are described, in particular, the shear-tension failure. The influence of various factors on the shear strength of simply supported reinforced concrete beams is described. Results of exploratory tests at the University of Manchester are discussed, with particular reference to the comparison of the strength of rectangular, T-, and L-beams. The behavior of beams loaded through secondary beams is studied on the basis of new tests.

■ THE RESISTANCE OF REINFORCED CONCRETE BEAMS to shear has been studied for nearly 60 years, and a number of design formulas and recommendations have been made from time to time. An excellent historical review of this work up to 1951 has been written by Hognestad,[1] and it would be superfluous to undertake such a review again. It may be of interest, however, to recall that the first known publication on web reinforcement, in 1899 by W. Ritter, introduced the principle of truss analogy and recommended stirrup design on the basis of that analogy — and the same formula and the same principle form, to this day, the basis of a large number of specifications for shear reinforcement.

There is little doubt that the actual behavior of reinforced concrete beams does not conform to the assumption of the standard theories in use. Moreover, the current design method, as far as shear is concerned, may lead, in some cases to structures with a factor of safety of less than two. This unsatisfactory state of affairs has been long realized, and was accentuated by the introduction of ultimate strength design methods into many codes of practice, for the design of the same beam according to

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the plastic method in flexure and elastic method in shear is untenable. Furthermore, shear failure is often sudden and occurs with little warning so that knowledge of the behavior of reinforced concrete beams failing in shear is important.

Experimental investigation of shear requires a considerable amount of time and material, and in smaller laboratories it is possible to study only a part of the problem. Fortunately, however, the available literature on shear tests contains a wealth of information, which, where sufficiently full data are given, can be studied and reinterpreted in the light of new ideas. The present paper uses the data of American and British tests and also some older German tests in just this way, in addition to information obtained direct from tests by the authors, and the action of shear reinforcement is discussed. In particular, the action of internal forces and their redistribution in the cracked section of a beam, leading to its ultimate failure, are studied; while redistribution of internal forces in beams without web reinforcement has been long realized, the continuous redistribution of internal forces from the time of formation of the first diagonal tension crack in a beam *with* web reinforcement appears not to have received sufficient attention.

In the first part of the paper beams without web reinforcement will be studied, not because they are common — indeed, they are not considered desirable — but because in them the action of shear can be, in the first instance, studied more easily. Various forms of web reinforcement both in rectangular and in L- and T-beams will be studied in the succeeding parts of the paper to explain the action of shear and to establish the most efficient and reliable system of reinforcement which would prevent the failure of a reinforced concrete beam in shear.

Notation

A_s = cross-sectional area of tension reinforcement
a = shear span
b = width of rectangular beam, or web width of T-beam
C = resultant (inclined) compression force in concrete
C' = horizontal component of the compression force in concrete
d = effective depth of beam
f_c' = compressive strength of concrete test cylinder
f_{cu} = compressive strength of concrete test cube
jd = lever arm of internal forces in beam
kd = depth of compression zone of concrete

Σo = sum of perimeters of bars in tension reinforcement
P = total load on beam
P_c = total load on beam at formation of first diagonal tension crack
P_u = total load on beam at failure
$p = \frac{A_s}{bd} \times 100$
T = force in main tension reinforcement
T_h = horizontal component of T
T_v = vertical component of T
V = total shearing force
V_c = shearing force resisted by concrete above neutral axis
v = nominal shearing stress in concrete
v_c = nominal shearing stress at formation of first diagonal tension crack
v_u = nominal shearing stress at failure

CURRENT METHODS OF DESIGN FOR SHEAR

It is not proposed to describe the present-day methods of design for shear since they are well-known, and indeed practiced. There are, of course, some differences between the codes used in various countries but a more important difference lies in the philosophy of the approach to shear, i.e., whether a beam should be designed so that failure in shear can never occur, or alternatively whether the mode of failure is unimportant as long as the factor of safety is sufficient. This is an important and far reaching problem which will be discussed later in the paper. Whatever the attitude to this problem, all current codes, except the 1955 Russian code, base the calculations of the shear strength of a beam under the working load on the permissible shearing stress, v, given by a formula of the type:

$$v = \frac{V}{b\,jd} \qquad (1)$$

where V = shearing force at the section considered
jd = lever arm
b = breadth of the beam in the case of rectangular beams and the breadth of the web for T-beams

It may be observed that according to the various codes the shearing strength of a T- or L-beam is taken to be no greater than that of a rectangular beam whose breadth is equal to the breadth of the web of the T-beam. The experimental evidence on this point will be discussed further in the paper.

It might be expected that the provision of web reinforcement in accordance with the requirements of the various codes ensures a satisfactory resistance of a reinforced concrete beam to shear, but this is not the case. Compliance with the code is not a sufficient guarantee of such a resistance, and a variable factor of safety results. This factor of safety is more likely to be low in beams with light tension reinforcement, while the code requirements for heavily reinforced beams often tend to be conservative. The strength of concrete is by no means the only factor

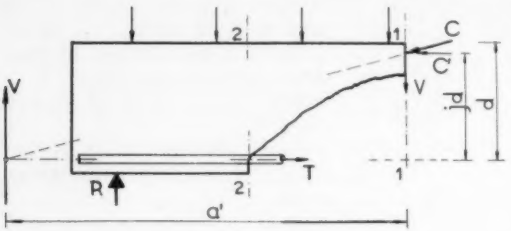


Fig. 1—Free body diagram immediately after formation of diagonal tension crack

that affects the strength in shear of a reinforced concrete beam without web reinforcement; the tension reinforcement and the a/d ratio are equally important factors.

A further criticism arises from the fact that the code regulations are based on the assumption that vertical stirrups resist axial tension only, and likewise, the main tension reinforcement is subjected to a horizontal tension only. It is apparent from the physical distribution of strains, as exemplified by the angle between the diagonal tension crack and the vertical stirrups, and also between this crack and the tension steel, that the resultant stresses in both the stirrups and the main steel must consist of tensile and shearing components. Hence, it can be seen that not only concrete and the stirrups resist the shearing force on the section but the main reinforcement plays its role too.

Furthermore, the action of concrete and of the stirrups (or bent-up bars) is such that their resistance to shear is not simply additive; in particular, the stirrups and the bent-up bars come into action only after the concrete has cracked. This makes for difficult calculations. In the succeeding parts of the paper the arrangement of web reinforcement which ensures the development of the full flexural strength of the beam will be discussed. However, beams without web reinforcement will be studied first to understand better the action of shear in a reinforced concrete beam.

REDISTRIBUTION OF INTERNAL FORCES

Shear failure occurs in beams in which the main tension reinforcement and the concrete in the compression zone are strong enough to resist the stresses induced by the moment applied while, as a result of the shearing force applied, severe diagonal tension cracking takes place.

The diagonal cracking may lead to failure in a variety of ways. The diagonal tension crack generally opens first in the middepth of the beam, either forming independently (usually when the ratio of the shear span to the effective depth, a/d, is small) or as a development of an existing flexural crack which is extended in an inclined direction (generally for larger values of a/d ratio). When the load applied increases in magnitude the diagonal tension crack increases in length and width. As the lower

end of this crack reaches the main tension reinforcement the balance of internal forces alters; this was first observed by Mörsch,[2] and more recently was discussed by Moody, Viest, Elstner, and Hognestad.[3]

Let us consider the part of a beam extending from the diagonal tension crack and a vertical section through the concrete above the crack to the near support. Fig. 1 shows the free body diagram immediately upon the formation of the diagonal tension crack but before it has widened. At this stage, the force in the tension reinforcement, T, still acts in a horizontal direction. For static equilibrium the tension in the steel, the compression in the concrete, and the resultant external load must all meet at a point. Likewise, the external moment applied must be equal to the moment of the internal forces:

$$Va' = Tjd$$

whence

$$T = \frac{V\,a'}{jd} \qquad (2)$$

It is, therefore, apparent that once the diagonal tension crack has reached the level of the tension reinforcement the force in it at Section 2-2 depends upon the moment at Section 1-1, i.e., the distribution of forces in the tension reinforcement along the beam no longer follows the distribution of external moments. This change is called a redistribution of internal forces.

Upon further increase in the applied load the diagonal tension crack may extend at either end, leading ultimately to the failure of the beam. This failure will eventually take place by the destruction either of the compression zone above the diagonal crack or of the tension zone below it. The former type of failure is known as shear-compression failure and the latter type will, by analogy, be referred to as shear-tension failure. This has sometimes been ignored, although Ferguson[4] stated that "the shear-compression theory alone has not adequately described the complete failure process observed in many tests."

The influence of various factors on the shear resistance of reinforced concrete beams has been studied by one of the authors in the concrete laboratory of the University of Manchester for a number of years.[5] Some of these tests will be referred to in detail further in the paper; at this stage only the photographs of typical shear failures are shown. Beam R7, shown in Fig. 2, is an example of a rectangular beam failing in shear-compression while the T-beam, T34, shown in Fig. 3, failed in shear-tension. Both these beams were reinforced with plain round bars with hooks.

Beam III-27b shown in Fig. 4 of Part 1 of a paper by Moody, Viest, Elstner, and Hognestad[3] is a typical example of shear-compression failure of a rectangular beam with straight deformed bars, and Fig. 3 of Ferguson's 1956 paper[4] shows a shear-tension failure of a rectangular

Fig. 2 — Shear-compression failure of rectangular beam

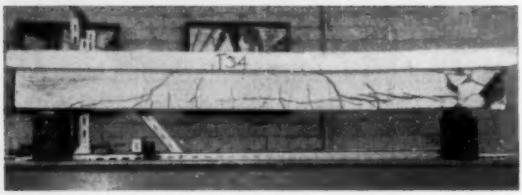


Fig. 3—Shear-tension failure of T-beam

beam with straight deformed bars. Examples of a simultaneous shear-compression and shear-tension failure are beams of Series B of Moody, Viest, Elstner, and Hognestad.[3]

TYPES OF SHEAR FAILURE

There appear to be four distinct types of shear failure; the punching or "pure" shear is not considered within the scope of the present paper.

In shear-compression failure the extension of the upper end of the diagonal tension crack reduces the size of the compression block to such an extent that the compressive stress in concrete becomes equal to its ultimate strength; crushing then takes place. This type of failure has been recognized for some time and has been described in detail (e.g., see Fig. 2).

The ultimate strength of a beam failing in shear-compression clearly depends on the compressive strength of concrete but is less affected by the tension reinforcement; the latter influences the moment of inertia of the beam and thus its deflection: the lower the deflection the less the diagonal tension crack widens and hence the less destructive its action.

In the case of the shear-tension failure the widening of the lower portion of the diagonal tension crack leads to a deformation of the main

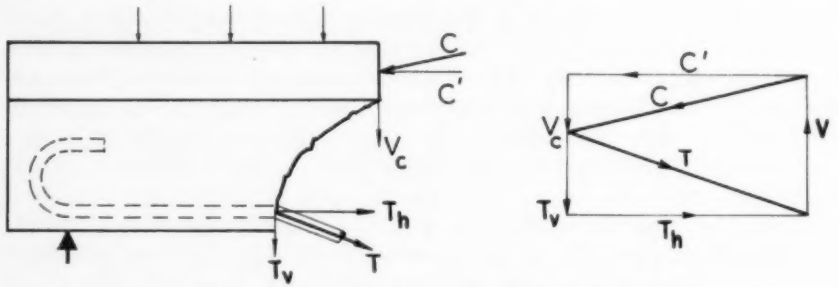


Fig. 4—Free-body diagram after widening of the diagonal tension crack

reinforcement; the two parts of the beam to either side of the crack rotate relative to one another approximately about the apex of the crack. This causes the tension reinforcement in the crack to be pressed down so that the force in that part of the reinforcement which crosses the diagonal crack is no longer horizontal. Fig. 4 shows the free-body diagram and the polygon of forces acting on the body at this stage of loading.

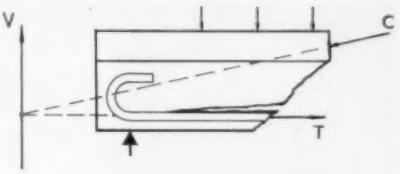


Fig. 5—Splitting at the level of the tension reinforcement

The vertical shearing force in the main steel, T_v, aided by the horizontal force T_h, causes the formation of a horizontal crack along the tension steel. This crack extends gradually toward the near support and, as a consequence, the inclination of the tension steel on the support side of the diagonal tension crack becomes smaller. For this reason the magnitude of T_v decreases. An experimental proof of this has recently been afforded by the tests of Watstein and Mathey,[6] who found that the maximum shear transferred by the tension steel occurred under a load of less than half the ultimate load, and beyond that decreased rapidly. For static equilibrium to be preserved, the decrease in the inclination of the steel must be accompanied by an increase in the inclination of the compressive force in concrete. This stage is shown in Fig. 5.

The splitting of the concrete at the level of the steel destroys the bond progressively up to the support. Thus the force in the tension steel between the original diagonal tension crack and the near support is constant, and, therefore, considerably in excess of the value calculated by the elastic theory. For equilibrium, the compressive force in concrete at sections between the diagonal tension crack and the support increases, too. The distance between the line of action of the resultant compression and the tension in steel is smaller the nearer the section considered is to the support.

In this manner the regime of forces changes from that in a flexural member to a two-hinged tied arch. This is illustrated in Fig. 6, and has been confirmed experimentally in various beams subjected to a high load. Evans[7] found that, in many cases, the horizontal stress has become entirely compressive along certain vertical sections while at other sections the compressive stress has changed to tension. Jones[8] also observed tensile

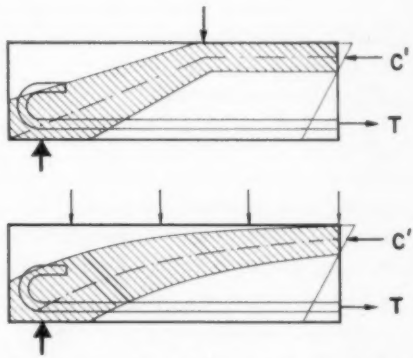


Fig. 6—Regime of forces after splitting

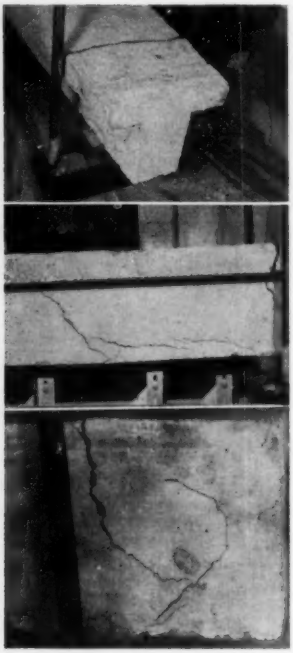


Fig. 7—Cracking in the top of the beam over supports. Top—Top view of T-beam T3. Middle—Side view of T-beam T3. Bottom — Rectangular Beam R3

stresses in the top fibers near the supported ends of some beams under loads close to the ultimate. Similar behavior has been noted by the authors;[5] the resulting cracks at failure can be seen in Beams R3 and T3, Fig. 7. It would therefore be expected that the absence of this tension zone in the top of the beam does not affect the shear failure; this is shown by Ferguson's test on a beam with a cut-away top half over the support.[9]

An experimental confirmation of the "tie-rod" behavior can be obtained by a study of some of the tests of Bernaert and Siess[10] on simply supported beams subjected to a uniformly distributed load. They measured the strain in the tension reinforcement at different points over its entire length at various stages of loading. For example, in Beam D-17 the strain distribution along the tension steel was parabolic, i.e., followed the bending moment until the appearance of the first diagonal tension crack. When this crack opened and reached the level of the tension steel, the strain in this steel increased rapidly from the intersection with the crack up to the support, but the strain distribution between the crack and midspan remained approximately parabolic. This change in the stress in the tension steel toward the support appears to confirm the progressive change in the distribution of internal forces.

An additional confirmation of the tied arch action after the diagonal tension crack has been fully developed is obtained from the measurement of concrete strain by Watstein and Mathey,[6] and analyzed by these authors.[11]

Once the bond between the steel and the concrete has been destroyed right up to the beam support, failure takes place in one of the following ways. If no mechanical anchorage is provided at the end of the bars they generally slip out and the beam can no longer carry any load. This was the case in the beam of Ferguson's [4] Fig. 3 mentioned earlier. Where the bars end in hooks these usually destroy the ends of the beam, thus causing collapse, as for instance in Beam T34 tested by the authors (Fig. 3).

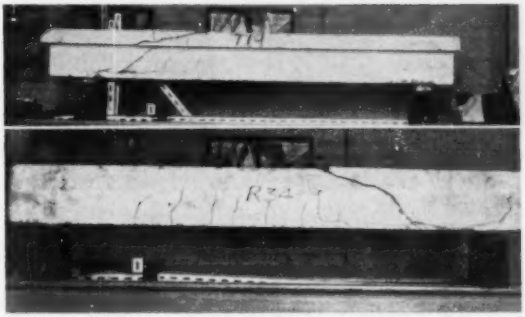


Fig. 8—Splitting failure of (top) T-beam T19 and (bottom) rectangular Beam R34

The third type of failure, which occurs particularly when mechanical bond and anchorage are strong, is for the diagonal tension crack to continue at a decreased slope through the compression zone up to the top surface of the beam, thus causing a total separation. The flat-slope crack opens suddenly, and can occur either independently or simultaneously with the horizontal splitting of the beam at the level of the tension steel, i.e., with the shear-tension failure.

The flat-slope crack opens when the principal tensile stress in the compression zone exceeds the tensile strength of concrete. The magnitude and direction of the principal tensile stress depend on the shearing force in the concrete above the crack. A full analysis of these stresses was made by Ferguson,[4] who considers this form of failure to be a special case of the shear-tension failure.[12]

An example of the flat-slope failure is shown in Fig. 8a of Beam T19. In this beam the diagonal tension crack first opened at middepth of the beam, making an angle of 45 deg with the axis of the beam. This crack extended approximately to the underside of the flange and also downward to the level of the tension steel. A redistribution of internal forces now took place but because the crack was nearer to the support than in Beam T34 (Fig. 3) the tension force in the main steel was lower, and despite the development of the horizontal split the beam end was not destroyed. The flat-slope crack opened suddenly and the beam collapsed. Beam R34, shown in Fig. 8b, failed in a similar manner: the vertical movement of the part of the beam above the crack can be seen clearly; there is no sign of crushing of the concrete.

What appears to be a simultaneous failure in shear-tension and flat-slope separation can be seen in Al-Alusi's[13] Fig. 3, who tested beams with end anchorage plates. A similar behavior is apparent in Beam B56B4 with external yokes, tested by Morrow and Viest.[14] Ferguson also observed a "separation complete both at top and bottom" of beams with anchorage plates, but did not find it possible to establish the primary failure pattern. This can also be seen in Fig. 9 showing Beam T13.

There is a further type of shear failure, which takes place when the shear span is extremely small. In this case the failure is due to the crushing of a concrete strut connecting the load point with the support, as

Fig. 9—Simultaneous shear-tension and flat-slope splitting failure of Beam T13

observed by Morrow and Viest.[14] According to Mattock,[15] the effective width of the strut depends primarily on the width of the loading plate and the ratio of the shear span to the effective depth.

FACTORS INFLUENCING SHEAR FAILURE

The question arises which of the various types of failure described above is likely to take place in a given beam. The factors governing this, as well as the load carrying capacity of the beam, are believed to be:

(1) Ratio of shear span to effective depth of beam.
(2) Shape and proportions of beam.
(3) Cross-sectional area of main tension and compression reinforcement expressed as a fraction of the section area of concrete.
(4) Size of tension bars and their method of anchoring.
(5) Compressive strength of concrete.
(6) Arrangement of loading and the magnitude of the bending moment and shearing force applied.

Ratio of shear span to effective depth of beam

The shear span-effective depth ratio, a/d, has a decisive influence on the mode of shear failure of reinforced concrete beams. The strut-like failure with a very low a/d ratio, perhaps less than 1, was described earlier. When the a/d ratio exceeds 1 but is small the diagonal tension cracks are directed from the point of application of the load to the support in almost a straight line. With increase in load the crack extends direct into the compression zone and shear-compression failure takes place.

When a/d is larger usually one diagonal tension crack forms, either developing first at middepth of the beam or, more often, from a flexure-tension crack. With increase in load the lower end of the crack extends, leading to splitting of the concrete along the main tension steel. In this manner the compression zone is only little reduced and the beam continues to take compression until failure. If because of local weakness of concrete the crack occurs rather near the support, the upper end of the diagonal tension crack may extend at a flattening slope. Thus the actual failure may occur either by the destruction of the tension zone at the beam end or by a rather sudden opening of the crack from the beam end right up through the compression zone. The latter occurs generally when mechanical anchorage is provided at the ends of the tension reinforcement.

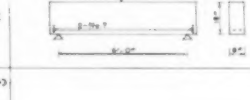
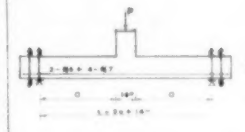
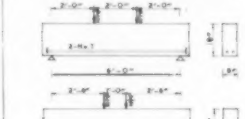
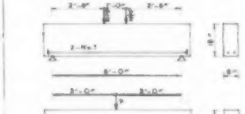
Let us now consider the influence of the a/d ratio on the shear capacity of a beam in the case of the shear-compression failure. Here, the larger the shear span the larger the bending moment under the load point for a given applied load. If the beam depth is constant, for a higher a/d

ratio there is thus a higher compressive stress in the concrete; consequently a smaller load will make this stress reach the crushing strength of concrete.

In the case of the shear-tension failure, an increase in the a/d ratio also means a higher compressive force in concrete under the load point for a given applied load. After the diagonal tension crack has extended to the tension reinforcement the force in it at the bottom of the crack increases as well, being always equal to the compression in concrete. Thus the higher the a/d ratio the larger the force in steel and, therefore, the lower the load carrying capacity of the beam.

Some tests illustrating the influence of the a/d ratio are summarized in Table 1. In Clark's[16] tests the a/d ratio was varied by a change in the distribution of the load points, the length of the span being kept constant. On the other hand, in some of the tests by Morrow and Viest[14] the loading

TABLE 1—INFLUENCE OF a/d RATIO ON SHEAR STRENGTH OF BEAMS WITHOUT WEB REINFORCEMENT

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f'_c psi	P percent	P_c kips	v_c psi	$\frac{v_c}{f'_c}$	P_u kips	v_u psi	$\frac{v_u}{f'_c}$
Clark[16]	D0-1,3		1.17	3755	0.98				100	450	0.121
	C0-1,3		1.56	3500	0.98				76.7	351	0.100
	B0-1,2,3		1.95	3435	0.98				51.5	236	0.069
	A0-1,2		2.34	3445	0.98				44.3	203	0.059
Morrow and Viest[14]	B14B4(*)		0.96	3820	1.85	100	328	0.086	225	741	0.194
	B21B4		1.45	3930	1.85	80	263	0.067	178	586	0.149
	B28B4		1.93	4690	1.85	60	197	0.042	115	380	0.081
	B56B4		3.86	3950	1.85	55	181	0.046	55	181	0.046
	B84B4		5.80	3950	1.88	50	166	0.042	50	166	0.042
	B113B4R		7.79	4160	1.86				38	125	0.030
	B14B2		0.96	2120	1.85	70	230	0.108	165	543	0.256
	B21B2		1.45	2010	1.86	70	231	0.115	107	354	0.176
	B28B2		1.93	2130	1.88	55	184	0.086	90	300	0.141
	B56B2		3.86	2130	1.85	45	148	0.070	45	148	0.070
	B70B2		4.82	2370	1.86	40	132	0.056	40	132	0.056
	B14B6		0.96	6780	1.85	100	328	0.048	350	1150	0.170
	B21B6		1.45	6600	1.82	100	323	0.049	260	849	0.127
	B28B6		1.93	6360	1.85	80	263	0.041	145	477	0.075
	B56B6		3.86	6630	1.83	61.5	202	0.030	61.5	202	0.030

Note (*) First number gives length of shear span a in in.

arrangement was kept constant (one load at midspan) but the span was varied, thus altering the a/d ratio. Their beams were restrained by vertical yokes placed at and just outside the supports with the object of preventing the horizontal splitting of the beam (known to develop in the shear-tension failure). These beams cover a wide range of a/d ratios — from 0.96 to 7.79 — and for this reason include a variety of modes of failure. From the photographs in their paper it can be seen that the beam with the a/d ratio of 0.96 failed in a strut-like manner. The beam with the a/d ratio of 1.45 failed in shear-compression. The beam with the a/d ratio of 1.93 exhibited what appears to be a balanced failure between shear-compression and shear-tension: the concrete crushed near the top of the diagonal tension crack and splitting along the main steel took place. The beam with a/d of 3.83 failed suddenly in shear-tension when splitting along the main steel occurred. The highest a/d (7.79) led to a flexural failure by the yield of steel at mid-span.

Mattock[15] is of the opinion that the shearing force V decreases less than the increase in a so that the value of the product Va decreases until it reaches a limit value. He plotted Va against a/d and showed that there is a relative maximum value of Va for a value of a/d of between 1 and 2,

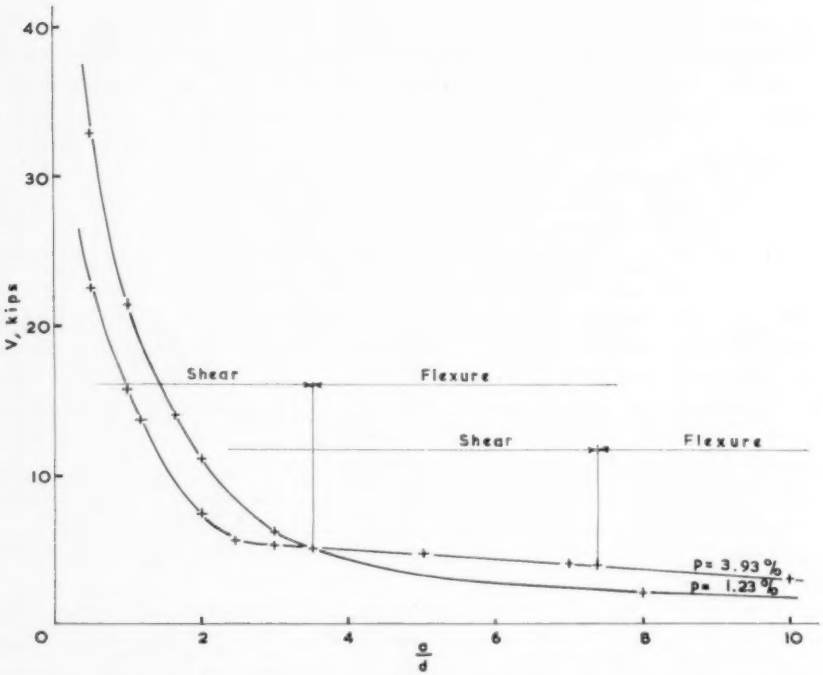


Fig. 10—Shearing force at failure plotted against a/d ratio for Mattock's data[15]

with a relative minimum for a/d between 2 and 4, depending on the percentage area of the tension steel. This is borne out by Morrow and Viest's results, as shown in Fig. 8 of their paper.[14]

If, however, instead of plotting the bending moment we plot the shearing force against the a/d ratio for Mattock's[15] tests, we obtain a graph in the form of a rectangular hyperbola, as shown in Fig. 10. This is not unlike the corresponding graph of Morrow and Viest.[14] Wilby[17] also found a similar shape of the plot of V against a/d, both for rectangular and T-beams. It may be interesting to note that by virtue of the properties of a hyperbola, $1/V$ against a/d would plot as a straight line.

This relation between V and a/d means that beams with very low a/d ratios have a very high strength in shear. This has been verified experimentally by Morrow and Viest[14] and by Mattock,[15] the failure being similar to the compression failure of a strut. Beams with very small values of a/d are really at the boundary of true flexural members and, as such, will not be discussed in more detail.

From the above discussion it can be seen then that the strength in shear of a beam depends not only on the shearing force applied but *also on the bending moment*. This statement can be verified by considering the shear failure of beams subjected to a uniformly distributed load, as done in the section on the influence of the arrangement of loading.

Shape and proportions of beam

The effect on shear of the depth to breadth ratio of the beam has been investigated by Bach and Graf[18] who tested T-beams differing only in the web widths: 5.9, 7.9, and 11.8 in.; the main reinforcement consisted of plain bars. All these beams developed diagonal tension cracks directed toward the load point; these cracks caused the pressing down of the tension steel resulting in splitting of the concrete and a slip of steel, i.e., a typical shear-tension failure. The initial cracking loads for all the beams were such that the nominal shearing stresses, v_c, were the same for all the beams. Likewise, the ultimate loads gave the same nominal shearing stress, v_u, for all the beams. In other words, the loads were directly proportional to the width of the web.

A similar comparison is available from Ferguson and Thompson's tests.[9] For instance, Beams D2 and A1 differed only in the web widths, which were in the ratio of 1.75 The ultimate shears of these beams were in the ratio of 1.79.

There is little information available on the comparison of the shear strength of T-beams and rectangular beams and, as mentioned earlier, the design for shear is based on the assumption that the strengths in shear of T-beams and of beams of rectangular section are equal to one another when the breadth of the latter beam is equal to the breadth of the web of the T-beam.

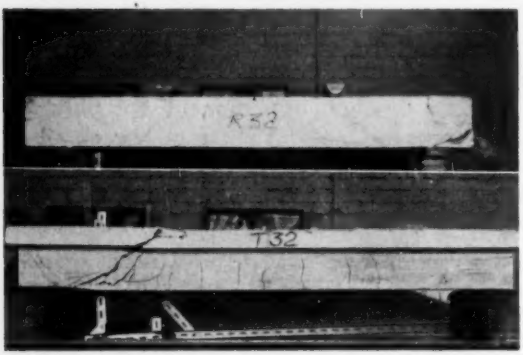


Fig. 11—Failure crack patterns. Top—Destruction of beam end in Beam R32. Bottom—Splitting along diagonal tension crack in Beam T32

The actual behavior of T-beams differs increasingly from that of rectangular beams as the flange area increases, for comparable web widths. The primary function of the flange is to provide a stronger compression zone, and for this reason failure occurs generally in shear-tension and not shear-compression, even for small a/d ratios. Tests on a number of corresponding pairs of rectangular and T-beams have been made at the University of Manchester, and the test data and results are given in Table 2.

All the beams had plain round mild steel reinforcement (of the type commonly used in England) with hooks; the yield stress varied between 36,000 and 40,000 psi. High-early-strength portland cement was used, and the aggregate was of siliceous origin with a maximum size of ⅜ in. The water-cement ratio varied between 0.48 and 0.67 with the aggregate-cement ratio between 4 and 6 so as to obtain medium workability. The strength of concrete in each beam is listed in Table 2. Compaction was achieved by casting the beams on a vibrating table; after stripping at 24 hr they were cured in a fog room at 20 C for 6 days, and then stored in the laboratory for 24 hr, at the end of which period they were tested.

The loading arrangement is shown diagrammatically in Table 2. The loads were applied through 2-in. rollers and the supports acted through 2-in. half-rounds except for some of the initial tests when a 2-in. bearing plate was interposed between the half-round and the bottom surface of the beam. One support rested on rollers so as to allow longitudinal movement. Typical photographs of beams of Table 2 are shown in Fig. 2, 7, and 11.

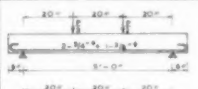
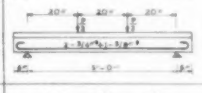
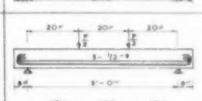
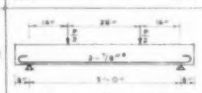
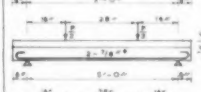
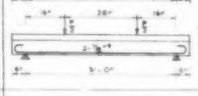
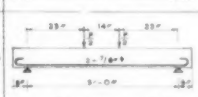
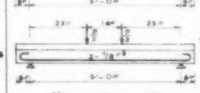
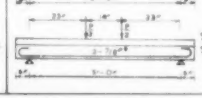
From Table 2 it can be seen that the ultimate strength of T-beams is between 13 and 61 percent higher than the strength of the corresponding rectangular beams, all beams failing in shear. The investigation of this problem, including tests on L-beams, is continuing, but an indication of the higher shear strength of T-beams, as compared with rectangular beams, can be obtained from Table 2. Some complication arises from the

fact that the plane of bending and the principal plane of an L-beam do not coincide.

Percentage area of main tension and compression reinforcement

The main tension steel contributes materially to the shear resistance of the beam and the role of the main steel is greater in the case of shear-tension failure, i.e., generally for high a/d ratios. This is illustrated by Table 3, which shows a considerable increase in the shear capacity of a beam when p is increased for a/d ratios between 3 and 5. When a/d is less than 2 the effects of increase in p are small or nonexistent. The effect on the cracking load seems to be similar to the effect on the ultimate load.

TABLE 2—COMPARISON OF SHEAR STRENGTH OF RECTANGULAR, T-, AND L-BEAMS

Beam No.	BEAM DETAILS	$\frac{a}{d}$	f'_{cu} (**) psi	p percent	P_c kips	v_c psi	$\frac{v_c}{f_{cu}}$	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	P_u as a ratio of P_u for rectangular beams	Mode of failure
R 3		2.91	4775	3.80	8.0	165	0.035	16.0	330	0.069		Destruction of beam end by hooks
T 3		2.91	4480	3.60 (*)	9.0	186	0.042	19.0	392	0.088	1.19	Destruction of beam end by hooks
R 7		2.76	3680	3.40	9.2	182	0.049	12.5	246	0.067		Shear-compression
T 4		2.76	3480	3.40 (*)	11.0	217	0.062	20.0	394	0.113	1.60	Destruction of beam end by hooks
R 32		2.37	5950	4.46	8.7	184	0.031	19.5	412	0.069		Destruction of beam end by hooks
T 32		2.37	6680	4.46 (*)	16.6	351	0.052	22.0	465	0.070	1.13	Splitting of beam along diagonal tension crack
L 32		2.37	5120	4.46 (*)	11.2	237	0.046	22.4	474	0.092	1.15	Destruction of beam end by hooks
R 34		3.41	4400	4.46	9.4	199	0.045	11.0	233	0.053		Splitting of beam along diagonal tension crack
T 34		3.41	6020	4.46 (*)	11.2	237	0.039	17.7	375	0.062	1.61	Destruction of beam end by hooks
L 34		3.41	5500	4.46 (*)	9.0	190	0.035	14.3	303	0.055	1.30	Destruction of beam end by hooks

Note: (*) Based on width of stem
(**) Measured on 3" cubes

Al-Alusi's[13] tests on T-beams with a/d ratios between 4 and 8 do not confirm the influence of the percentage area of main reinforcement observed by other investigators. The reason for this is not obvious.

The dependence of the influence of the tension reinforcement on the value of the a/d ratio is believed to confirm indirectly the suggestion regarding the relation between the mode of failure in shear (shear-compression or shear-tension) and the a/d ratio.

The influence of the compression reinforcement is less clear. In his early tests Wilby[17] found the load capacity of rectangular beams in shear to decrease with increase in the percentage area of compression reinforcement. He ascribed this somewhat unexpected phenomenon to the high horizontal shearing stress in the concrete at the level of the compression steel, with consequent rapid horizontal splitting of the concrete at that level, and release of bond. The beams tested had a/d ratios between 2.1 and 3.2.

Wilby's tests on T-beams, with the a/d ratio between 2.1 and 6.2, showed that the percentage area of compression reinforcement was of no influence on the load carrying capacity of the beams in shear.

Bernander's[19] tests on rectangular beams with a/d ratio of 4 indicate that compression reinforcement has no significant effect on the shear strength of beams with high tensile reinforcement. This was also found by Al-Alusi.[13]

On the other hand, Laupa, Siess, and Newark's[20] analysis has shown the compression reinforcement to contribute to the shear strength of rectangular beams both without and with web reinforcement. Jones'[8] tests on beams with inclined stirrups show that compression reinforcement carries a considerable proportion of vertical shear, and thus indicates that in beams with this type of web reinforcement the compression reinforcement contributes to the shear strength of the beam.

It can be seen that it is not possible to state conclusively the effect of compression reinforcement on the shear strength of a beam. The authors are conducting experimental work on this effect and the results to date[5] indicate that compression reinforcement does not increase the shear strength of a beam.

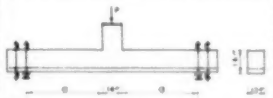
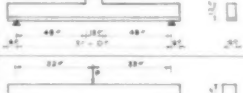
Size of tension bars

Moody, Viest, Elstner, and Hognestad[3] have shown that when the same steel area is provided by more than one bar there appears to be some increase in both the cracking and the ultimate loads. This should be considered as no more than a tentative conclusion. The increase in strength of these beams, which were 7 in. wide, seems to occur when two or three bars are used instead of one, but when four bars are used in one layer there appears to be no increase in load, and there is possibly even some decrease.

The following explanation is tentatively offered by these authors. The formation of a crack in the vicinity of the tension steel is connected with the destruction of bond between the steel and the concrete over a certain distance on either side of the crack. The use of a larger number of smaller bars for the same area of steel means that the bond stress is smaller, and hence the resistance to bond failure is higher. This would explain a higher cracking load and also a higher ultimate load in this case. It may be observed that in Moody, Viest, Elstner, and Hognestad's tests that the cracking load was nearly equal to the ultimate load.

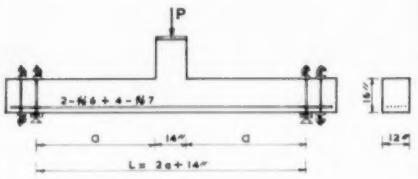
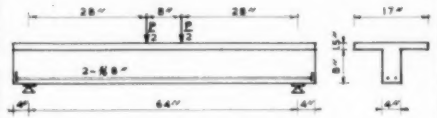
On the other hand, the larger the sum of the diameters of the bars the smaller the net cross section of the concrete (through the plane of the bars). Thus the area of concrete resisting its splitting is smaller. This would seem to explain why there is an optimum number of bars to be

TABLE 3—INFLUENCE OF PERCENTAGE AREA OF TENSION REINFORCEMENT ON SHEAR STRENGTH OF BEAMS WITHOUT WEB REINFORCEMENT

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$ (*)	f'_c psi	p percent	Ratio of values of p	P_c kips	P_u kips	Ratio of values of P_c for high and low p	Ratio of values of P_u for high and low p
Moody, Viest, Elstner and Hognestad [3]	III-24a		1.52	2540	2.72		40	133		
	25b		1.52	2500	3.46	1.27	45	130	1.12	0.98
	III-27a		1.52	3100	2.72		45	156		
	29a		1.52	3150	4.25	1.56	60	175	1.33	1.12
Morrow and Viest [16]	B14B6 (**)		0.965	6780	1.85		100	350		
	B14A6		1.00	6590	3.83	2.07	125	405	1.25	1.16
	B21B6		1.42	6600	1.82		100	260		
	B21A6		1.50	6570	3.83	2.10	99	260	0.99	100
	B28B6		1.93	6360	1.85		80	145		
	B28A6		2.02	6840	3.83	2.07	95	150	1.19	1.03
	B56B6		3.82	6630	1.83		61.5	61.5		
	B56A6		4.00	5780	3.79	2.07	80	80	1.30	1.30
Gaston, Siess, Newmark [9]	T2Ma		3.40	4320	1.38			18.5		
	T2Mc		3.40	4470	1.90	1.38		25		1.35
Laupa, Siess, Newmark [10]	S-1 (***)		4.79	3940	1.46			16.8		
	S-2		4.54	3900	2.08	1.42		19.1		1.14
	S-5		4.66	4330	4.11	2.82		22.4		1.17
Smith [20]	37a,b		4.4	5240	1.65		6	7		
	18a,b		4.0	5090	3.07	1.86	8.5	9.5	1.42	1.36
	40a,b		4.4	5450	4.42	2.68	7.5	10.7	1.25	1.53
Evans [8]				1:1:2 mix	1.1			4.1m (**)		
					1.7	1.55		6.4m		1.56
					2.1	1.91		7.8m		1.90
				1:2:3 mix	1.1			3.4m		
					1.7	1.55		4.7m		1.38
					2.1	1.91		5.3m		1.56
				1:3:5 mix	1.1			2.3m		
					1.7	1.55		3.4m		1.48

Notes: (*) For stub beams a is measured to the edge of the stub.
(**) First number gives length of shear span a in in.
(a) $m \cdot a$ constant (unknown)
(***) For beam S-1: $a = 5$ in.

TABLE 4—INFLUENCE OF COMPRESSIVE STRENGTH OF CONCRETE ON SHEAR STRENGTH OF BEAMS WITHOUT WEB REINFORCEMENT

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f'_c psi	p percent	P_c kips	$\frac{v_c}{f'_c}$	P_u kips	$\frac{v_u}{f'_c}$	Ratio of values of $\frac{P_u}{f'_c}$ for high and low strength
Morrow and Viest[14]	B14B2(+)		0.965	2120	1.85	70	0.108	165	0.256	2.12
	B14B6		0.965	6780	1.85	100	0.048	350	0.170	
	B21B2		1.45	2010	1.86	70	0.115	107	0.176	2.43
	B21B6		1.42	6600	1.82	100	0.049	260	0.127	
	B28B2		1.96	2130	1.88	55	0.086	90	0.141	1.61
	B28B6		1.93	6360	1.85	80	0.041	145	0.075	
	B56B2		3.86	2130	1.85	45	0.070	45	0.070	1.37
	B56B6		3.82	6630	1.83	61.5	0.030	61.5	0.030	
Ferguson and Thompson[9]	A 2		3.40	3960	4.75(++)			121.2	0.053	1.25
	A 5		3.40	6580	4.75			152.2	0.040	

Notes: (+) First number gives length of shear span a in in.
(++) Based on width of stem

used. When this optimum is exceeded—and its value depends on the width of the beam—the positive effects of the decrease in bond stress are offset by the decrease in the resistance to splitting. For the 7 in. width of Moody's beams the optimum seems to be about three bars.

This behavior has been confirmed in tests at the University of Manchester,[5] and is now being studied more fully.

It has been mentioned earlier that when the horizontal splitting of the concrete at the level of the tension steel has extended up to the support, the force in the steel is of constant magnitude (and therefore considerably higher than would be calculated from the applied moments) over its length between the original diagonal tension crack and the support; the ability of the beam to take a further increase in load depends then on the anchorage of the main steel, as shown in Part 5 of this paper.

An experimental verification of the change in steel strains can be obtained from Mains'[21] tests, in which the strain in the main steel along its full length was measured in rectangular beams with various types of reinforcement; all beams were subjected to symmetrical two-point loading. In beams with plain bars and hooks subjected to a load over half the ultimate, the strain was approximately constant over the length of the shear span and the value of this strain was considerably in excess of the theoretical value.

From this it can be seen that the bond had been nearly destroyed over the length of the shear span, and the force in the bar in this portion of the beam was, therefore, transmitted direct to the hooks. Thus the presence of the hooks was essential for the beam to continue to take a further increase in load until failure.

The problems of bond and anchorage require a more detailed discussion, to which Part 5 of this paper is devoted.

Compressive strength of concrete

The influence of this factor is greatest when the failure is of the shear-compression type, and for this reason the effect of the compressive strength of concrete on the ultimate strength of a beam in shear decreases with increase of the a/d ratio. This is borne out by some of the test results of Morrow and Viest[14] on rectangular beams and also by Ferguson and Thompson's[9] tests on T-beams; these data are collected in Table 4.

It may be further observed that although v_u increases with increase in f_c' the ratio v_u/f_c' is lower for higher concrete strengths and may actually reach the allowable value of 0.03; in such beams a sudden failure can occur.

Arrangement of loading and the magnitude of the bending moment and shearing force applied

There seem to be two broad groups of loading arrangement as far as the shear failure of simply supported beams is concerned. Beams loaded by one or two point loads appear to belong to one group, while beams subjected to more than three point loads or to a uniformly distributed load form the second group.

In the first group one diagonal tension crack generally forms in such a position that the upper end of the crack points toward the point of application of the load. The lower end of the diagonal crack extends in the direction of the support; this extension may be either in the form of a nearly straight line continuation of the diagonal tension crack, or else the inclined crack may continue down to the level of the tension steel and hence along this steel to the support. Thus for the first group the failure occurs at the section subjected to both the maximum bending moment and the maximum shearing force.

When the loading consists of four or more point loads or is uniformly distributed, more than one diagonal tension crack in either half of the beam may be formed; some of these cracks may develop from initial flexure-tension cracks. The position of the diagonal tension crack that causes the ultimate failure is of interest: this critical crack can be seen in Fig. 12 representing one of the beams tested by Bach and Graf,[22] and also in some of Ferguson's[4] beams. It is important to note that the shear failure does not occur in the zone of maximum shear, as calculated in the usual manner.

This behavior can be explained, at least partly, by Mörsch's[2] observation that the actual shearing force distribution does not coincide with that calculated by the accepted theory. As the bending moment increases along the beam the neutral axis rises, with a consequent more rapid rise in the compressive stress. This is accompanied by a rise in the shearing stress and, consequently, also in the principal tensile stress. Thus the highest principal stress does not occur next to the support but at a section where the combined effects on the principal tensile stress of the bending moment and the shearing force are greatest.

Ferguson[4] compared the shear capacity of beams subjected to two and four point loads symmetrically disposed. All beams were of the same dimensions and a span of 60 in. Beam F2 had two loads 24 in. from the supports and failed under a shearing force of 5000 lb. In Beams F3 and F5 two additional loads acted within the original 24-in. shear span; their failing shears were 8290 and 7690 lb, respectively. The crack patterns of all these beams were similar, and therefore unaffected by the arrangement of loading. It can be noted that in all cases the top of the diagonal tension crack was at a section subjected to sensibly the same bending

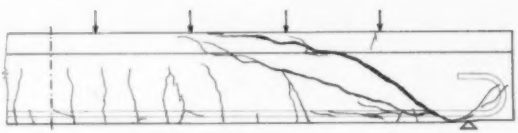


Fig. 12—Crack pattern of Beam 593[22] subjected to a uniformly distributed load: failure by splitting along the diagonal tension crack

moment, viz, 120, 139, and 135 in.-kips. In this manner the similarity in the behavior and the mode of failure of the three beams can be explained: the design criterion of shear only is clearly not correct.

From considerations of bending moment and shearing force diagrams Whitney[23] suggested that the equivalent shear span for uniformly distributed loading be calculated by the expression

$$a = \frac{1 - x^2}{2x} L$$

where xL is measured from the center of the span of 2 L.

The critical zone for shear in Whitney's tests is between 0.6 and 0.7 of the distance from the center of the span to the support.

The average position of the critical zone in Bernaert and Siess'[10] tests is similar to that of Whitney's, viz. 0.78L. Some beams tested by Bach and Graf[22] (e.g., Beam 593) had the critical crack in a similar position (Fig. 12).

The effects of the deflection of a beam on its ultimate strength in shear are still to be fully investigated. Although the influence of the deflection can be observed in the tests of many investigators no clear conclusions are available.

It should be remembered that the deflection is a function not only of the span, as discussed above, but also of the loading arrangement, i.e., the number and position of the load points. This would affect the ultimate shear capacity of a beam, and it is possible that this factor has also entered Ferguson's tests.[4]

SHEAR STRENGTH OF BEAMS LOADED THROUGH SECONDARY BEAMS

In all the tests described so far the loads were applied through plates directly onto the top surface of the beam, and the supports were similarly in contact with the bottom surface of the beam. When loads and supports act in this manner a local vertical compression may be induced in the beam. On the other hand, in the majority of full size structures the actual loads on a main beam are applied through the medium of secondary beams framing into the main beam. Attention to this was drawn by Ferguson[4] who suggested that the method of transfer of loads affects the shear strength of a beam. It is clear that such an influence, if it is a real factor, would be of considerable importance in the interpretation of test data.

TABLE 5— INFLUENCE ON SHEAR STRENGTH OF LOADING THROUGH SECONDARY BEAMS

Beam No.	Method of application of load	$\frac{a}{d}$ (+)	f_{cu} (++) psi	p percent	P_c kips	v_c psi	$\frac{v_c}{f_{cu}}$	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	P_0 as a percentage of beam loaded and supported direct.	Mode of failure
1, 2		2.09	2225	4.47	16.00	348	0.156	21.52	467	0.210	100	Destruction of beam end at direct support
3, 4		2.09	2090	4.47	13.80	300	0.144	18.64	405	0.194	87	Extension of diagonal tension crack in main beam into cross beam, destruction of beam end at direct support
6		2.09	2280	4.47	12.16	264	0.116	19.88	432	0.189	92	Extension of inclined cracks in cross-beam into main beam leading to destruction of beam end at direct support
7		2.09	2280	4.47	12.14	264	0.116	18.78	408	0.179	87	Extension of inclined cracks in supporting cross-beam into main beam, leading to beam collapse
5		2.09	3780	4.47	13.90	302	0.080	26.66	580	0.154		As No.7
8		2.09	3780	4.47	14.36	312	0.082	24.30	528	0.140		Extension of the upper end of diagonal tension crack at flat slope across the cross-beam, splitting of beam along diagonal tension crack at direct support

Notes: (+) a measured centre to centre of cross beams
(++) Measured on 5" cubes

In Ferguson's investigation[4] beams loaded directly were compared with similar beams supported and loaded by secondary beams integrally connected with the main test beam. All beams were rectangular and had no web reinforcement; they were subjected to third-point loading with the a/d ratio of 1.35, reckoned between centers of forces.

From the test results it appears that when the load was applied through the secondary beams, or when the beam was supported by the secondary beams, the ultimate load was 40 percent of the ultimate load of a similar beam loaded and supported directly. When both the loads and the supports acted through the secondary beams the strength of the beam was only 32 percent of the strength of a beam loaded and supported directly.

This large difference in strength was ascribed by Ferguson to the restraining effect of the direct vertical compression induced by the loads and the supports when applied direct to the surface of the test beam. As an indication of this restraint Ferguson described his earlier test[9] on a beam with a cut-away top half over the support: despite the weakened end section the diagonal tension crack opened further away from the support.

It should be noted that the a/d ratio of this beam was 3.4, and the position of the crack and the mode of failure were the same as those of beams with a similar a/d ratio, e.g., Beam T19 shown in Fig. 8. Furthermore, the cut-away part of the beam is subjected to tension (see Fig. 6 and 7) at loads nearing the ultimate, and therefore the absence of that part of the beam is immaterial. This behavior was discussed in detail earlier in this paper, in connection with the redistribution of internal forces.

It should be remembered that in Ferguson's beams the a/d ratio was only 1.35, and the restraining effect is not likely to extend along the beam more than a distance equal to half its effective depth, or possibly even less. A confirmation of this is obtained from Morrow and Viest's[14] tests on stub beams, in which the strains in the tension steel and on the compression surface of the concrete were measured along the shear span.

For example, in Beam B28E4 under a load of 45 kips, i.e., before the first diagonal tension crack has formed, the steel strains increased gradually from the support toward the face of the stub through which the load was being applied. The first diagonal tension crack formed at 55 kips, and failure took place under a load of 120 kips. When the load was 100 kips the measurement of strains showed that the stress in steel was constant over the entire length of the shear span. This means that horizontal splitting of concrete with a consequent loss of bond had taken place.

The strains in the concrete, under a load of 45 kips, increased gradually up to within some 2½ in. of the face of the stub (i.e., 1/6 of the

effective depth of the beam); a sudden increase in strain took place there, and the concrete strain was of higher value up to the face of the stub. Under a load of 100 kips the strains in concrete within 2½ in. of the face of the stub increased considerably and there was also some increase in concrete strain within the adjacent 3½ in., i.e., a total of 7 in. was affected.

It may also be interesting to note that under this load tension in the top surface existed within 15½ in. of the reaction.

In the vicinity of the end reactions, which supported the beam over a 4 in. length, no changes in steel or concrete strains were observed. The effective depth of the beam was 14.5 in., and it can, therefore, be seen that the restraining effect could be of influence only in beams with a very low a/d ratio.

An experimental verification of this is afforded by the authors' tests on rectangular beams without web reinforcement with the a/d ratio of 2.09. The beam details are shown in Fig. 13; the presence of stirrups in the cross-beams should be noted.

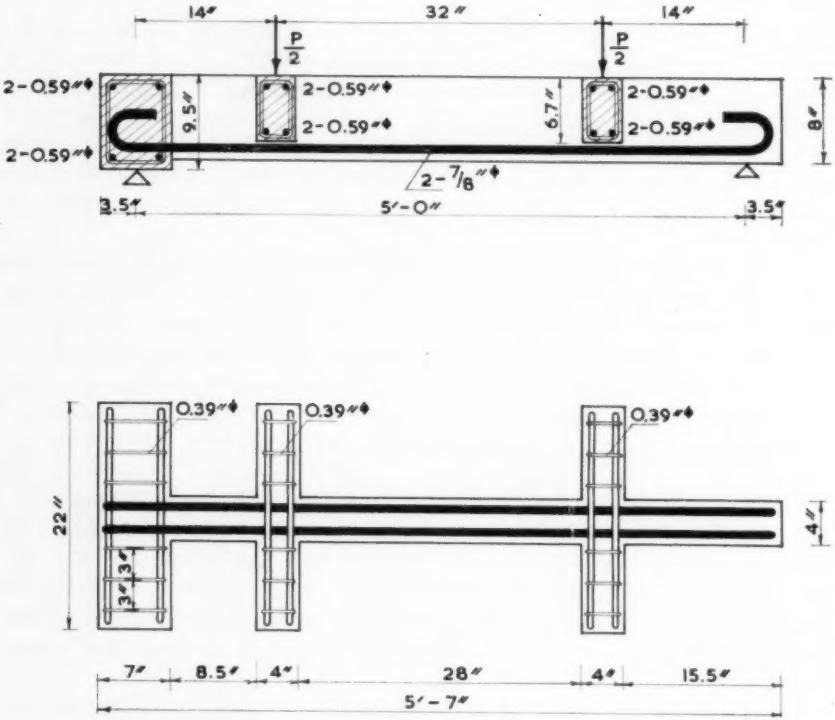


Fig. 13—Details of beams loaded through secondary beams

The results of the authors' tests are summarized in Table 5, and it can be seen that when the loads were applied through cross-beams the ultimate load on the beam was between 87 and 93 percent of the load on a similar beam subjected to direct loading. These figures are based on the results of two similar beams for each condition.

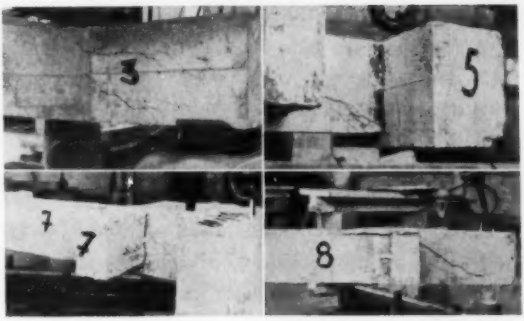


Fig. 14—Secondary beam loading series: Beams 3, 5, 7, and 8

Two additional beams were made of higher strength concrete, and nearly the same ultimate load was recorded for the beam loaded through the secondary beams (No. 8) as for the beam loaded direct (No. 5). The ultimate load of these beams was higher than that of beams made of weaker concrete. Fig. 14 shows Beams 3, 5, 7, and 8. Beam 7 shows the upper end of the crack and Beam 5 the lower end of a similar crack.

Of the four beams which were supported by secondary beams at one end and directly at the other, two failed in the part nearest to the cross-beam support (No. 5 and 7) while the other two beams failed near the direct support (No. 6 and 8). In particular, Beam 5 (Fig. 14), loaded directly, failed near the support through the secondary beams, while Beam 8 (Fig. 14), loaded through secondary beams, failed near the direct support. Since all beams were loaded symmetrically this behavior does not appear to confirm the inherent greater strength of beams supported directly, as compared with those supported by secondary beams.

It is possible, however, that the slightly lower strength of the beams with the secondary beams is due, at least in part, to the cracks in the secondary beams connecting with the diagonal cracks in the main beam. For this reason, it is interesting to observe the mode of failure near the secondary beam supports. This cracking of the secondary beams occurred despite their being reinforced by tension bars and by stirrups, and it is possible that the lack of stirrups contributed to the comparatively low strength of Ferguson's[4] beams loaded through the secondary beams. Thus the importance of a satisfactory connection of the beams is evident, and the provision of suitable reinforcement to prevent the development of inclined cracks in the supporting beams is essential.

SUMMARY AND CONCLUSIONS

It can be seen that the accepted formulas for the calculation of shearing stress in concrete are not based on the correct factors influencing

this stress, and, as a consequence, they result in a varying factor of safety in shear. It is thus possible for a reinforced concrete beam to fail in shear before its moment capacity has been exhausted, and shear failure is, owing to its sudden occurrence, undesirable.

The mode of formation of the diagonal tension crack has been described in detail, together with the redistribution of internal forces, which commences when this crack reaches the tension reinforcement. A further increase in load leads to shear failure in one of several ways.

In shear-compression failure, the extension of the upper end of the diagonal tension crack reduces the compression zone until crushing of concrete takes place.

In shear-tension failure, it is the lower end of the diagonal tension crack that extends, leading to splitting of the concrete at the level of the tension steel. The beam now behaves like a two-hinged tied arch until either the reinforcement slips or the end anchorage destroys the beam end. Alternatively, if the mechanical bond or anchorage are strong enough, after the horizontal crack has opened, the diagonal tension crack extends at a decreased slope through the compression zone up to the top surface of the beam, and splitting of the beam along the diagonal tension crack takes place.

When the shear span is very small a strut-like failure takes place.

From a discussion of the various factors affecting the type of failure and the load carrying capacity of the beam, the paramount importance of the a/d ratio can be seen. This factor, determining largely whether failure will occur in shear-compression or whether the generally slower shear-tension failure will take place, is completely neglected by all current codes.

The contribution of the tension reinforcement to the shear capacity of a beam and the influence of anchorage have been discussed. The role of anchorage and the problems of bond in general will be discussed more fully in Part 5 of this paper.

Some tests on the influence of the arrangement of loading have been described, and it was shown that under uniformly distributed loading, failure in shear does not occur at the point of maximum shearing force.

Some results of the authors' tests have shown that the flange in T- and L-beams affects the strength of the beam in shear; this is another factor not recognized by the codes.

The authors' tests on beams loaded through secondary members have shown that a good connection between the main beam and cross-beams is essential; otherwise, premature failure in shear may take place. The effects of vertical restraint at load points are believed to be limited to beams with a very low a/d ratio.

Finally, it is interesting to note that no lesser an authority than Whit-

ney[23] suggested that the diagonal tension cracking load, and not the collapse load, be considered as the ultimate load on a beam without web reinforcement, for purposes of design. This is, according to Whitney, justified because "the integrity of a beam without shear reinforcement is lost when diagonal cracking occurs, and repeated loading may cause failure."

Since the diagonal tension cracking load is generally much lower than the ultimate load in flexure, and also to avoid failure without warning, it would be advisable, in the opinion of these authors, to provide web reinforcement in *all* beams. This argument is pursued in Part 4 of this paper.

ACKNOWLEDGMENT

The authors would like to express their thanks for assistance in the experimental work: Beams 1 to 8 of Table 5 were tested at the Building Research Station, Technion, Haifa, Israel; the other beams were tested in the concrete laboratory of the University of Manchester, many of them by E. Lord.

REFERENCES

Reinforcement in Concrete," *Circular Series* No. 64, University of Illinois Engi-

1. Hognestad, E., "What Do We Know About Diagonal Tension and Web neering Experiment Station, 1952.

2. Mörsch, E., *Der Eisenbentonbau*, 6th Edition, Konrad Wittwer, Stuttgart, 1929.

3. Moody, K. G.; Viest, I. M.; Elstner, R. C.; and Hognestad, E., "Shear Strength of Reinforced Concrete Beams," ACI JOURNAL, V. 26 (*Proceedings* V. 51): "Part 1 —Tests of Simple Beams," No. 4, Dec. 1954, pp. 317-332; "Part 2—Tests of Restrained Beams Without Web Reinforcement," No. 5, Jan. 1955, pp. 417-434; "Part 3—Tests of Restrained Beams with Web Reinforcement," No. 6, Feb. 1955, pp. 525-539; "Part 4—Analytical Studies," No. 7, Mar. 1955, pp. 697-730.

4. Ferguson, Phil M., "Some Implications of Recent Diagonal Tension Tests," ACI JOURNAL, V. 28, No. 2, Aug. 1956 (*Proceedings* V. 53), pp. 157-172.

5. Neville, A. M., and Lord, E., "Tests on Shear Failure of Reinforced Concrete Beams," *Concrete Laboratory Report*, University of Manchester, 1958.

6. Watstein, D., and Mathey, R. G., "Strains in Beams Having Diagonal Cracks," ACI JOURNAL, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), pp. 717-728.

7. Evans, R. H., "Experiments on Stress Distribution in Reinforced Concrete Beams," *The Structural Engineer* (London), V. 14, Mar. 1936, pp. 118-130.

8. Jones, R., "The Ultimate Strength of Reinforced Concrete Beams in Shear," *Magazine of Concrete Research* (London), V. 8, No. 23, Aug. 1956, pp. 69-84.

9. Ferguson, Phil M., and Thompson, J. Neils, "Diagonal Tension in T-Beams Without Stirrups," ACI JOURNAL, V. 24, No. 7, Mar. 1953 (*Proceedings* V. 49), pp. 665-676.

10. Bernaert, S., and Siess, C. P. "Strength in Shear of Reinforced Concrete Beams under Uniform Load," *Civil Engineering Studies, Structural Research Series* No. 120, University of Illinois, June 1956.

11. Taub, J., and Neville, A. M., Discussion of Reference 6, ACI JOURNAL, V. 30, No. 12, June 1959 (*Proceedings* V. 55), pp. 1454-1457.

12. Ferguson, Phil M., Author's closure to discussion of Reference 4, ACI JOURNAL, V. 28, No. 12, June 1957 (*Proceedings* V. 53), pp. 1190-1193.

13. Al-Alusi, A. F., "Diagonal Tension Strength of Reinforced Concrete T-Beams with Varying Shear Span," ACI JOURNAL, V. 28, No. 11, May 1957 (*Proceedings* V. 53), pp. 1067-1077.

14. Morrow, JoDean, and Viest, I. M., "Shear Strength of Reinforced Concrete Frame Members Without Web Reinforcement," ACI JOURNAL, V. 28, No. 9, Mar. 1957 (*Proceedings* V. 53), pp. 833-869.

15. Mattock, Alan H., Discussion of Reference 14, ACI JOURNAL, V. 29, No. 6, Part 2, Dec. 1957 (*Proceedings* V. 53), pp. 1352-1353.

16. Clark, Arthur P., "Diagonal Tension in Reinforced Concrete Beams," ACI JOURNAL, V. 23, No. 2, Oct. 1951 (*Proceedings* V. 48), pp. 145-156.

17. Wilby, C. B., "The Strength of Reinforced Concrete Beams in Shear," *Magazine of Concrete Research* (London), No. 7, Aug. 1951, pp. 23-30.

18. Bach, C., and Graf, O., "Versuche mit Eisenbetonbalken zur Ermittlung der Widerstandsfähigkeit verschiedener Bewehrung gegen Schubkräfte. Zweiter Teil," *Bulletin* No. 12, Deutscher Ausschuss für Eisenbeton, 1911 (published by W. Ernst & Sohn, Berlin).

19. Bernander, K. G., "An Investigation of the Shear Strength of Concrete Beams Without Stirrups or Diagonal Bars, Reinforced with High Tensile Steel with Various Rib Patterns," *RILEM Symposium on Bond and Crack Formation in Reinforced Concrete*, Stockholm, 1957, pp. 211-214.

20. Laupa, A.; Siess, C. P.; and Newmark, N. M., "The Shear Strength of Simple Span Reinforced Concrete Beams Without Web Reinforcement," *Civil Engineering Studies, Structural Research Series* No. 52, University of Illinois, Apr. 1953.

21. Mains, R. M., "Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars," ACI JOURNAL, V. 23, No. 2, Nov. 1951 (*Proceedings* V. 48), pp. 225-252.

22. Bach, C., and Graf, O., "Versuche mit Eisenbetonbalken zur Ermittlung der Widerstandsfähigkeit verschiedener Bewenhrung gegen Schubrkäfte. Dritter Teil," *Bulletin* No. 20, Deutscher Ausschuss für Eisenbeton, 1912, (published by W. Ernst & Sohn, Berlin).

23. Whitney, Charles S., "Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members Without Shear Reinforcement," ACI JOURNAL, V. 29, No. 4, Oct. 1957 (*Proceedings* V. 54), pp. 265-298.

24. Gaston, J. R.; Siess, C. P.; and Newmark, N. M., "An Investigation of the Load-Deformation Characteristics of Reinforced Concrete Beams up to the Point of Failure," *Civil Engineering Studies, Structural Research Series* No. 40, Univeristy of Illinois, Dec. 1952.

25. Smith, R. B. L., "The Influence of Shear on the Moment of Resistance of Reinforced Concrete Beams," paper read before Lancashire and Cheshire Branch of the Institution of Structural Engineers (England), Mar. 1958.

26. Evans, R. H., "Stresses in the Steel Reinforcement of Reinforced Concrete Structures," *The Structural Engineer* (London), V. 13, Sept. 1935, pp. 354-369.

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Strains in Ultimate Strength Design of Reinforced Concrete Beams

By G. C. REYNOLDS*

IT HAS BEEN POINTED out[1] that when using the formula in ACI 318-56 for the ultimate strength design of slabs and beams with compressive reinforcement, considerable under-estimation of the strength of the members can result. The method suggested for overcoming this difficulty seems to be unnecessarily complicated, and does not work for Examples 1 and 2 which follow.

The first of these examples is one in which the Code formula *overestimates* the strength of the member by 30 percent. The fact that the Code formula can overestimate the strength of a member is serious and some revision of the Code seems to be necessary. The difficulty with the Code formula seems to stem from the exclusion of strains in calculations. The use of strains is well known,[2] and it seems likely that it has been avoided because it involves a trial and error procedure. However, what seems to have been overlooked is that in cases where the Code formulas are satisfactory, all reinforcement yields before failure, and a first guess, that all reinforcement yields, will prove correct, as it does in Example 3. Consequently, calculations for these cases are simple. In cases where the reinforcement does not yield, a consideration of strains is then necessary to obtain reliable results.

When strains are considered, it is found that the same simple method can be applied to both reinforced and prestressed concrete beams and columns. In the case of columns, considerable time and effort can be saved if the simple principles, rather than formulas, are used.[3]

BASIC ASSUMPTIONS

Two examples will be worked out by using statics and the assumption that strains vary linearly across a section. Empirical factors will be used to obtain the resultant compressive force in the concrete and the position of this force. These factors are the

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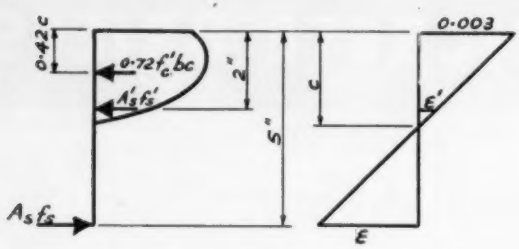

Fig. 1—Forces and strains at failure

factors implied by the Code formulas and by the usual equivalent rectangular stress block.[1] However, as it is not necessary to specify the shape of the stress block or the maximum stress, this is not done. It will be assumed that failure will occur when the maximum compressive strain in the concrete reaches 0.003.

Example 1

It is desired to find the ultimate moment a 5 in. wide strip, from a slab 6 in. deep, may withstand. This slab is reinforced with 1 sq in. of tensile steel at a depth of 4 in. and 0.6 sq in. of compression steel at a depth of 2 in.; $f_c' = 3000$ psi and the yield stress for steel is 60,000 psi. Although this slab is heavily reinforced it is still within all Code restrictions and could be considered as being used where there are limitations on dead load or headroom, or for a heavily loaded section of an otherwise lightly loaded slab.

The forces and strains at failure are shown in Fig. 1. The force in the concrete, $0.72\ f_c'\ bc$, is in this case $10{,}800c$ lb for $f_c' = 3000$ psi and $b = 5$ in. The depth to the neutral axis is c.

As a first try, guess that $c = 3$ in. From the strain diagram:

$$\varepsilon' = \frac{(4-3)}{3}\ 0.003 = 0.001$$

$$\therefore\ f_s' = 0.001 \times 30 \times 10^6 = 30{,}000 \text{ psi}$$

$$\varepsilon = \frac{(3-2)}{3}\ 0.003 = 0.001$$

$$\therefore\ f_s' = 30{,}000 \text{ psi}$$

assuming Young's modulus $= 30 \times 10^6$ psi.

Check horizontal equilibrium

$$10{,}800\ c + 0.6 \times 30{,}000 = 1 \times 30{,}000$$

$$\therefore\ c = 1.1 \text{ in., which is too small}$$

Guess that $c = 2.7$ in.

$$\varepsilon = \frac{(4-2.7)}{2.7}\ 0.003 = 0.00145$$

$$\therefore\ f_s = 43{,}500 \text{ psi}$$

$$\varepsilon' = \frac{(2.7-2)}{2.7}\ 0.003 = 0.00078$$

$$\therefore\ f_s' = 23{,}400 \text{ psi}$$

Check horizontal equilibrium

$$10{,}800\ c + 0.6 \times 23{,}400 = 1 \times 43{,}500$$

$$\therefore\ c = 2.7 \text{ in.}$$

To find the ultimate moment, moments are taken about the compression steel.

$$M = 43{,}500 \times (4-2) + 10{,}800 \times 2.7\ (2 - 0.42 \times 2.7)$$

$$M = 112{,}000 \text{ in.-lb}$$

The Code formula for this case is

$$M = (A_s - A_s')\ f_y\ d\ [1 - 0.59(p - p')\ f_y/f_c'] + A_s' f_y (d - d')$$

and this gives $M = 145{,}000$ in.-lb.

The error is 33,000 in.-lb or 30 percent and the formula is wrong because it assumes all the steel yields before failure.

Example 2

It is desired to find the ultimate moment for a 1 ft wide strip of slab 7 in. deep. This slab is reinforced with 0.6

sq in. of mild tensile steel at a depth of 5 in. with an equal quantity of compression steel at a depth of 2 in.; $f_c' = 3000$ psi and $f_s = 40{,}000$ psi. The Code formulas give an ultimate moment of 72,000 in.-lb if the top steel is considered as compressive reinforcement, and 110,000 in.-lb if the top steel is neglected.

Fig. 1 shows the forces and strains at failure. The force in the concrete, $0.72\ f_c'\ b\ c$, is in this case $26{,}000\ c$ lb.

As a first try guess that $c = 2$ in. so that ε and f_s' are zero. From the strain diagram, $\varepsilon = (5-2)/2 \times 0.003 = 0.0045$. This is greater than the yield strain so that $f_s = 40{,}000$ psi and $A_s f_s = 24{,}000$ lb.

From statics

$$26{,}000\ c = 24{,}000$$

$\therefore c = 0.92$ in., which is too small compared to the 2 in. assumed

As a second try guess that $c = 1.5$ in., which gives a value for ε greater than the yield strain and $A_s f_s = 24{,}000$ lb as before. The top steel has a tensile strain of $(2 - 1.5)/1.5 \times 0.003 = 0.001$ so that $A_s' f_s' = 0.6 \times 0.001 \times 30 \times 10^6 = 18{,}000$ lb tension.

From statics

$$26{,}000\ c = 24{,}000 + 18{,}000$$

$\therefore c = 1.6$ in., which is a little high

A third guess that c is 1.55 in. is satisfactory and gives $A_s' f_s' = 16{,}000$ lb tension.

To find the ultimate moment, M_u, moments are taken about the resultant force in the concrete.

$$M_u = 24{,}000(5 - 0.42 \times 1.55) + 16{,}000(2 - 0.42 \times 1.55)$$

$$M_u = 127{,}000 \text{ in.-lb}$$

This moment is considerably greater than the moments of 72,000 in.-lb and 110,000 in.-lb obtained by using the Code formulas, and quite clearly the Code formula which assumes the top steel yields in compression is considerably in error.

Example 3

Consider the same slab as before except with the top steel omitted.

As a first try guess that $f_s = 40{,}000$ psi; then as the force in the concrete is equal to the force in the steel

$$26{,}000\ c = 0.6 \times 40{,}000$$

$$\therefore c = 0.92 \text{ in.}$$

The strain is $(5-0.92)/0.92 \times 0.003 = 0.013$ which is well above the yield strain and so the guess is correct. It may be noted that at this strain most steels would have entered the work hardening range, which would give a reserve of strength.

$$M_u = 0.6 \times 40{,}000\ (5 - 0.42 \times 0.92)$$

$$M_u = 110{,}000 \text{ in.-lb.}$$

This is, of course, the same result as is given by the ACI Code formula. The calculation requires the same amount of work using statics as the Code formula, although in a slightly different form. The time required to calculate strains is trivial and merely replaces the calculation of the upper limit to the quantity of reinforcement associated with the Code formula.

REFERENCES

1. Guillard, Eugene, "Ultimate Resisting Moment of Beams with Compression Reinforcement," ACI JOURNAL, V. 29, No. 9, Mar. 1958 (*Proceedings* V. 54), pp. 759-765.

2. ACI-ASCE Committee 327, "Ultimate Strength Design," ACI JOURNAL, V. 27, No. 5, Jan. 1956 (*Proceedings* V. 52), pp. 505-524.

3. Gregory, M., and Reynolds, G. C., "Ultimate Strength Design of Reinforced Concrete Members under Combined Bending and Axial Load," *Commonwealth Engineer* (Melbourne), V. 46, No. 9, Apr. 1959.

Concrete's Etymological Offspring

By PETE CRETE

THE PHILOLOGICAL BONANZA noted in the November, 1958, JOURNAL* shows no sign of petering out. On the contrary, here are more nuggets off the old cube. As usual, they range from the sublime to the less so to the anything but; however, de gustibus, de Madison Avenue nil nisi bonum . . .

The sun never sets
on the admen's mad rush
to coin a new phrase
from CON-CRETE so lush.

So let us gather
these nuggets so pure
and print them in JOURNAL
to forever endure.

The terms "Guncrete" and "Stelcon" represent new uses of names already on the list, whereas the word "Stress-Con" is a new version (spelling) for an old term.

Word	Where used
Armor-Crete	(Coating for precast concrete) Preco Chemical Corp., New York; ACI JOURNAL, Oct. 1958 News Letter, p. 36
Conblox	(Precast concrete units) Conblox, Inc., New Orleans, *Lone Star Cement News*, Nov. 1959, p. 7
ConCreLese	(Coating to prevent concrete sticking to forms) Thomas Co., The, Minneapolis; *Contractors & Engineers*, May 1959, p. 173
Concretius	(Telegraph address) Concrete Publications, Ltd., London; subscription billheads
Conmix	Conmix (S. A.) Ltd., S. Australia; ACI JOURNAL, News Letter, Aug. 1959, p. 28 (under Barclay)
ConSeal	(Concrete joint sealing tape) Presstite - Keystone Engineering Products Co., St. Louis; *Engineering News Record*, Jan. 29, 1959, p. 9
Crete-Cure	(Curing membrane) Thiem Products, Inc., Milwaukee; *Civil Engineering*, June 1958, p. 133; ACI JOURNAL, News Letter, Oct. 1958, pp. 33, 35
Cretestone	(Concrete bonding plaster) Gyproc Products Ltd., Gravesend, Kent, England; *Engineering*, Sept. 18, 1959, adv. p. 76
Densicon	(Concrete floor protection) Maintenance Inc., Wooster, Ohio; *Civil Engineering*, Sept. 1958, p. 151
Epoxicrete	(Patching compound) Coast Pro-Seal & Mfg. Co., Los Angeles; *Engineering News-Record*, Sept. 17, 1959, p. 230
Epoxy-Crete	(Floor surfacing and patching compound) Howe & French, Inc., Weymouth, Mass.; ACI JOURNAL, Mar. 1959, News Letter, p. 30
Ferricrete	(Soil horizon with a layer of concretions in which ferrica is the cementing agent; "Ort-

*Previous lists appeared in ACI JOURNAL: *Proceedings* V. 47, Nov. 1950, p. 263, May 1951, p. 752; *Proceedings* V. 49, Mar. 1953, pp. 682-684; *Proceedings* V. 50, June 1954, pp. 894-897; *Proceedings* V. 51, Apr. 1955, pp. 813-817; *Proceedings* V. 53, Aug. 1956, pp. 226-228; *Proceedings* V. 54, Aug. 1957, pp. 166-167; *Proceedings* V. 55, Nov. 1958, p. 656.

stein" in German*); *American Scientist*, Dec. 1958, pp. 361, 366

*cf Silcrete; loc. cit., pp. 360, 366

Fibrecrete (Lightweight floating concrete dock) Fibrecrete Corp., Long Beach, Calif.; ACI JOURNAL, Feb. 1958, News Jetter, p. 33

Forticrete (Exposed-aggregate wall panels) Forticrete Ltd. (England); *Engineering News-Record*, Apr. 16, 1959, p. 64

Guncrete (Moldable and castable refractory) Morgan Refractories Ltd., Neston, Wirral, Cheshire; *The Engineer*, May 1, 1959, adv. p. 33

Kaocrete (Castable refractory) Babcock & Wilcox, New York; *Iron Age*, Feb. 19, 1959, p. 73

Maraconcrete (Concrete made with Maracon admixture) Marathon, Menasha, Wis.; *Engineering News-Record*, May 7, 1959, p. 10

Nox-Crete (Anti-bond form coating) Industrial Lubricants, Inc., Omaha; ACI JOURNAL, Nov. 1959, News Letter, p. 21

Plazcrete (Plasticiser) Sealocrete Products Ltd., London; *Concrete and Constructional Engineering*, Oct. 1959, adv. p. xlv

Pneukret (Pneumatic concrete placer) The Concrete Pump Co., Ltd., London; *Concrete and Constructional Engineering*, Oct. 1957, adv. p. v.

Pres-Crete Missouri Pres-Crete, Inc., Overland, Mo.; ACI JOURNAL, Feb. 1958, News Letter, p. 18

Ready-Con (Saw cart) Eveready BrikSaw Co., Kansas City, Mo.; ACI JOURNAL, Dec. 1957, News Letter, p. 36

Speed-Crete (Additive for quick-setting grout) Underwater Problem Engrg. Corp., Ft. Lauderdale, Fla.; *Engineering News-Record*, Feb. 26, 1959, p. 40

Stelcon (Precast concrete bulkhead sections) *The Engineer*, Apr. 11, 1958. p. 531 (col. 1 top)

Stress-Con (Name of prestressing division) Shute Concrete Products Co., Inc. Richmond, Ind.; *Concrete*, Nov. 1959, p. 13

Stresscrete (Prestressed concrete) Sumter Stresscrete, Inc., Sumter, S.C.; Eau Claire Stresscrete, Inc., Eau Claire, Wis.; Leaflet listing Leap Associate producers of prestressed concrete (May, 1959; Stresscrete Constructions Pty Ltd., Rose Bay, NSW, Australia; ACI JOURNAL. Aug. 1957, News Letter, p. 25 (under Antill)

Terracrete (Soil-cement houses, New Zealand) *Engineering News-Record*, Jan. 29, 1959, p. 48

Voidcrete (Forming system for precast products) Elgood Concrete Forms, Corp., Brooklyn, N. Y.; sales literature

Walcrete (Masonry cement) Cement Marketing Co., Ltd., London; *Concrete and Constructional Engineering*, Aug. 1958, p. 296

And two possible duplications (although the magazine write-up of the first calls it "original"):

Autocrete (Automatic concrete plant) Millars' Machinery Co. Ltd., Bishops Stratford, Herts, England; *Civil Engineering and Public Works Review*, Dec. 1958, p. 1421

Formcrete (Contractors) Formcrete Ltd., London; *Concrete and Constructional Engineering*; Oct. 1957, adv. p. xlv

Visual Determination of Size Fractions in Sand Sieve Analysis

By W. J. BOOTH*

When making a sieve analysis of fine aggregate, the U. S. Bureau of Reclamation's *Concrete Manual*† states, p. 404, paragraph (e), "A visual arrangement of sizes should be made to provide a better conception of the grading." I have developed a device to give an indication of the grading of sand commonly used in concrete mixtures without having to refer to figures in print or to rely on memory. The device, as illustrated in Fig. 1, is for a 500-g sample and is calibrated by weighing samples of sand of each size fraction corresponding to the specified maximum and minimum limits and placing them into their respective compartments. A line is drawn on the back of the compartment, or on the glass, corresponding to each limit for each fraction.

To use the device, the sand on each screen is weighed as usual when determining the fineness modulus, but instead of emptying each fraction onto a bench or into a separate pan, it is poured into its respective compartment. A visual inspection shows the amount of each fraction and immediately determines whether or not it lies within its specified limits.

The device is simple to make from readily available materials.

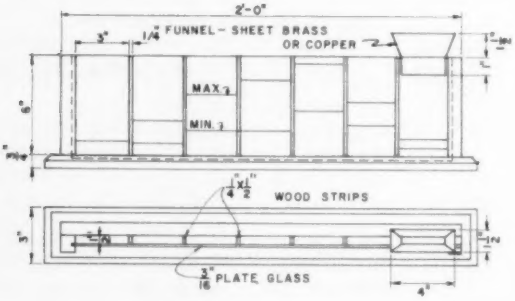

Fig. 1 — Device for visual determination of size fractions in sieve analysis

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†6th Edition (Revised reprint), U. S. Department of Interior, Bureau of Reclamation, Denver, 1956, 492 pp.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Prestressed bridge with a heated deck

Contractors and Engineers, V. 57, No. 5, May 1960, pp. 58-59, 62-63

An electrical snow melting system was built into the deck of this 1311-ft viaduct in Aberdeen, S. D. The heating elements were imbedded in the concrete beneath the wheel tracks in the four traffic lanes. There are three 1/4 in. copper tubes containing the heating elements under each wheel track, set about 5 in. apart and 1 1/4 in. below the surface.

Lightweight concrete was used in the deck as well as in the prestressed concrete girders. Instead of steel bearing plates, neoprene pads support the girders.

The 20 bents of the bridge are made up of prestressed concrete piles rising to a cast-in-place cap.

Some recent railway bridge renewals

R. T. Holloway, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 85 (Session 1958-59), pp. 57-88

Author's Summary

Some statistics are given in regard to the bridges and their upkeep on the railway system of Coras Iompair Eireann. The stress recording instruments used in load tests on bridges and some results obtained with them are described briefly. In deciding on the type of bridge, the method of erection and time factor are often decisive influences in railway work. The importance of a carefully planned erection scheme is emphasized and some points are listed which will require attention, especially if cranes are to be used for erection.

The remainder of the paper consists of brief descriptions of some of the bridges erected in the past 4 years. Those described are short or medium span structures of different construction, comprising two reinforced concrete road bridges, two post-tensioned precast reinforced concrete structures, three prestressed concrete road bridges, one reinforced concrete footbridge, three riveted and two welded steel girder type under-bridges.

Swan River Narrows Bridge

Engineering (London), V. 188, No. 4883, Nov. 20, 1959, pp. 506-507

Deck structure of the Narrows Bridge, Perth

The Engineer (London), V. 208, No. 5417, Nov. 20, 1959, pp. 648 and 655

Reviewed by Aron L. Mirsky

This 1100 ft long structure at Perth, Western Australia, has many unusual features. It has five spans (160, 230, 320, 230, and 160 ft) which are fully continuous under live load, a 70 ft wide roadway (six lanes) with an 8-ft sidewalk and 2-ft safety curb on either side, and carries many utilities.

The substructure is founded on long "Gambia" piles. Under the north shore pier a somewhat modified type was used, designed to permit possible move-

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ment in the reclaimed ground through which they were driven. The abutment design will also accommodate some settlement.

The superstructure utilized precast I segments jointed and stressed on staging erected in the river. To reduce the concrete thicknesses, the prestressing cable (19-wire strands of 0.7-in. wire) were placed outside the section, and encased in fine concrete and bonded to the beam webs by transverse mild steel shear bars after tensioning. An additional advantage of external cables is the greatly reduced loss of prestress due to friction. Vertical prestressing cables were used to reduce the principal tension stress due to shear in the thin webs; a one-third model was tested to verify the assumptions on which this design was based. The superstructure was designed and erected as double-cantilever beams with suspended spans during erection; the temporary hinges were then locked by caulking the joints with cement mortar and then clamping them together by short lengths of prestressing cable passing through ducts and overlapping the main cables, thus providing full continuity.

Suspended span is prestressed concrete

Engineering News-Record, V. 164, No. 16, Apr. 21, 1960, pp. 40-42

A novel prestressed concrete bridge spanning a canal in Belgium is a self-anchored suspension bridge. Basically the design is that of a three-span continuous beam with unreinforced concrete stiffeners and the prestressing cables located outside (above) the beam.

The bridge has a length of 590 ft with three suspended spans. The center span is 328 ft long and the two end spans are 131 ft. The bridge deck is 60 ft wide. The towers are independent of the roadway and its connections with the pier.

Contractor cast the concrete towers and their bracing in a position about 2 ft 4 in. below their final location then built up the endless cable by placing the untensioned wires around the ends of the bridge, in the cable clamps, and over the saddles on top of the towers. Workmen prestressed the entire superstructure by jacking up the towers alternately in pairs. After raising each column to its final position, cast-steel hinges were inserted under it which became permanent supports for the columns.

Highway bridges with stiffened edge members

ANDREW GALLIA, *Civil Engineering Transactions*, Institution of Engineers, Australia (Sydney), V. CE 2, No. 1, Mar. 1960, pp. 33-45
AUTHOR'S SUMMARY

Of highway bridges embodying stiffened edge members, the two extreme types are the slab bridge with low flexural transverse stiffness but full torsional stiffness, and the beam bridge with stiff transverse diaphragms but zero torsional stiffness. A factor, η, is defined for the purpose of characterizing the transverse distribution of wheel loads, and it is shown that for bridges with stiffened exterior girders an $\eta =$ 86 percent can be achieved.

Loading tests on two prestressed concrete highway bridges

ROY E. ROWE, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), Aug. 1959, pp. 477-498
Reviewed by ARON L. MIRSKY

Presents details and results of test on two short-span bridges under abnormal loads. First structure (St. Martins Bridge, Stamford, Lincolnshire) is composite-slab type; observed behavior is explained on basis that the "fill" concrete (the protective cover above the waterproofing membrane shaped to form the roadway camber) acted structurally to a certain extent, that there were edge-stiffening effects from the parapet beams, and that the structure acted as a beam-and-slab bridge with cracks between the precast beams. Second structure (Langstone Bridge, Hayling Island, Hampshire*) consists of precast prestressed beams laid side

by side, grouted, and prestressed transversely; author denotes it a "pseudo-slab-type," and reports good agreement between test results and theoretical values (by "load-distribution analysis") except for the "mean" deflection and the moments, which were lower than expected due to the edge-stiffening effects of the fascia beams which were tied to the roadway beams by the reinforced concrete sidewalk slabs.

*J. W. Melrose and W. A. Eyre, "Langstone Bridge, Hampshire," *Proceedings* ICE, V. 8, Oct. 1957, pp. 141-160; Current Reviews, ACI JOURNAL, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), p. 741

Concrete bridge goes up fast

Railway Age, V. 148, No. 16, Apr. 18, 1960, pp. 38, 45

Reviewed by ARON L. MIRSKY

Discusses the replacement of a bridge destroyed by fire in August, 1958. This bridge links the Westbound classification and the departure yards in the New Haven R.R.'s vital Cedar Hill (New Haven, Conn.) yard. It was rebuilt in 7 weeks during 1959 using prestressed, precast box beams on prestressed concrete piles. Advantages cited: speed, relative independence of winter weather, resistance to salt corrosion, and elimination of fire hazard.

Construction

Watertight parking deck

Contractors and Engineers, V. 57, No. 4, Apr. 1960, pp. 86-89

Placing the prestressed, precast concrete roof sections first and then casting the beams to support them was the unique sequence of operations on the construction of the exhibit hall portion of the Memorial Coliseum at Portland, Ore.

The heavy channel-shaped prestressed concrete sections that make up the roof deck were cast in a plant, trucked to the building site, and set in place on the falsework, which also supported the forms for the supporting beams and columns. When the concrete beams and columns were cast, they tied the entire 135 x 389-ft roof into a continuous unit. This is expected to minimize cracking and eliminate roof leaks.

Since it will have to take heavy wheel loads, the exhibit hall roof was designed for highway bridge loading, H2OS16 loading. The columns are spaced at 30-ft centers.

Construction of a twin Ctesiphon shell at the Central Building Research Institute

S. K. NARAYANA, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 429-433

AUTHOR'S SUMMARY

Describes the construction of a twin corrugated concrete shell. The design procedure is outlined, and the cost of construction is tabulated.

Northlight shell roofs — New constructional methods

IVO HRUBAN and KONRAD HRUBAN, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 423-426

AUTHORS' SUMMARY

Two types of shell roofs are described.

The first type is represented by a structure which is composed of a small number of large precast elements. The erection procedure is described and illustrated. A simplified method of computing and designing the shell member is given.

If local conditions shift the economic balance in favor of *in situ* construction, the cost of the structure will be considerably reduced if special movable formwork units of large size are used. This method has been applied to the second type of shell structure reported on.

Blackwall Tunnel improvements

Engineering (London), V. 187, No. 4867, June 19, 1959, p. 803

Reviewed by ARON L. MIRSKY

Blackwall Tunnel under the Thames, in the east end of London, was opened in 1897. Designed for horse-drawn and pedestrian traffic, it has long been inadequate, and a parallel tunnel will be constructed to afford a total of two lanes in each direction. Current con-

struction involves improvements at the north approach, including a composite bridge carrying the East India Dock Road over the approach and containing four 36-in. high pressure gas mains built within the deck, and a very heavy concrete invert in part of the approach to counteract the high water level.

New home for the Giants

RALPH MONSON, *Contractors and Engineers*, V. 56, No. 11, Nov. 1959, pp. 38-43

Describes the construction of the 40,000-seat stadium at Candlestick Point, San Francisco, which is to be the new home of the Giants.

The site for the horseshoe shaped stadium is a rock hill. As half of the lower deck seats were over the hill, it was graded to the shape of the lower-deck seats.

The upper deck consists of 41 big 12-ton A-frames which support the seat girders and cantilevered boomerang-shaped roof supports. Many other interesting building techniques are discussed.

Round-the-clock concrete operations

TONY MAVROUDIS, *Contractors and Engineers*, V. 57, No. 4, Apr. 1960, pp. 10-14

More than 3000 cu yd of concrete are placed daily for the Niagara Power Project generating plant. Concreting operations, batch plants, and aggregate storage are described. The contractor is using both prefabricated steel forms and timber forms fabricated on the job. The contractor is also producing 26,700 linear ft of prestressed concrete beams and 12,100 linear ft of cast-in-place diaphragms that will support a road across the top of the intake structure of the power house.

All-masonry fireproof school for $10.61 psf

Building Construction Illustrated, V. 28, No. 2, Aug. 1959, pp. 28-29

A brief resumé of construction techniques employed in the construction of a school. The cost of $10.61 psf was achieved by using precast interior bearing walls (rather than post and beam system), precast concrete plank floor system, and precast 4 x 4-ft hollow ring units for the foundation wall.

Heavy-weight concrete stabilizes tower frames

RALPH M. BAUMGARTEN, *Civil Engineering*, V. 29, No. 7, July 1959, pp. 39-41

Discusses the mix proportions, mix control during batching and placing, cost data, and erection or placing procedures of the heavyweight concrete which was placed at the base of two frame towers that were used to support a fully enclosed conveyor bridge. Heavyweight concrete was used because of space limitations, foundation conditions, and a high overturning moment.

Construction Techniques

Modern concrete tools and equipment

Building Construction Illustrated, V. 28, No. 2, Aug. 1959, pp. 40-42

Discusses the how, when, and where of concrete construction tools such as: vibrators, tools for wet finishing (power floats, trowels, and disc finishers), and grinding and chipping tools.

Northlight shell roofs — Practical aspects of construction

N. V. SHASTRI, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 427-428
AUTHOR'S SUMMARY

The construction aspect of reinforced concrete shell roofs does not always receive the attention it deserves. This paper deals with a few practical points on the fabrication, erection and removal of formwork, the placing of reinforcement, and the concreting of northlight shell roofs.

Treatments and coatings for concrete

Engineering (London), V. 188, No. 4888, Dec. 25, 1959, pp. 683-684
Reviewed by ARON L. MIRSKY

A review of recent developments in external treatment of concrete to improve its resistance to abrasion, water

penetration, or chemical attack. Five types of treatment are discussed: treatment with silica-forming media, "Ocrating" (treatment with silicon tetrafluoride), silicone treatment, tar epoxy coatings, and rubber coatings.

Concrete floor finishes for homes, offices, and public buildings

Concrete Association of India, Bombay, 1959, 52 pp., Re. 1

Several types of floor finishes are described and some good examples are given of terrazzo and tile finishes and the color and decorative designs that are possible. Appendices give detailed specifications for the various finishes in general use. Heavy duty floor finishes for industrial buildings are not covered.

Water-excluding structures

LEONARD R. CREASY and LAWSON S. WHITE, *Proceedings*, Institution of Civil Engineers (London), V. 14 (Session 1958-59), Sept. 1959, pp. 31-42

Reviewed by ARON L. MIRSKY

After a general discussion of the problem and the element of "good design," by both authors, there are separate notes by each author, reflecting his experience. White (pp. 34-36) discusses the membrane (asphalt mastic) type of waterproofing, while Creasy (pp. 36-42) discusses both membrane and nonmembrane types. Both authors are agreed on the importance of good structural form and of good workmanship, particularly in the concrete work.

Tension in concrete piles during driving

E. A. L. SMITH, *Journal*, Prestressed Concrete Institute, V. 5, No. 1, Mar. 1960, pp. 35-40

Author explains how tension can arise during driving of concrete piles and how its effects may be controlled. Long piles are more likely than short piles to develop horizontal tensile cracks. Horizontal tensile cracks usually appear in the upper half of the pile.

Horizontal tensile cracks may be reduced or eliminated by increasing the prestress or by gentle driving using short hammer strokes until such time as the pile point reaches firm resistance, after which full blows may be struck.

Dams

The Dokan project: The flood-disposal works and the grouted cut-off curtain

GEOFFREY M. BINNIE, JOSEPH G. CAMPBELL, NEVILLE H. GIMSON, PETER F. F. LANCASTER-JONES, and CYRIL A. GILLOTT, *Proceedings*, Institution of Civil Engineers (London), V. 14, (Session 1958-59), Oct. 1959, pp. 181-204

Reviewed by ARON L. MIRSKY

Part 1: The flood disposal works

G. M. BINNIE, J. G. CAMPBELL, and N. H. GIMSON, pp. 181-193

Describes briefly construction of two concrete lined tunnels, a diversion tunnel 12.5 m in (finished) diameter, about 300 m long, and a gated spillway tunnel 11 m in diameter and about 300 m long.

Part 2: The grouted cut-off curtain

P. F. F. LANCASTER-JONES and C. A. GILLOTT, pp. 193-204

Discusses reasons for specifying a grouted cut-off wall and grouting the left abutment, and methods used. Results to date indicate treatment has been effective.

The Dokan project: The dam

GEOFFREY M. BINNIE, JOSEPH G. CAMPBELL, RALPH H. EDGINTON, CHARLES A. FOGDEN, and NEVILLE H. GIMSON, *Proceedings*, Institution of Civil Engineers (London), V. 14 (Session 1958-59), Oct. 1959, pp. 157-180

Reviewed by ARON L. MIRSKY

After a brief outline of the project, paper covers the various factors (topography and geology of the site, and optimum shape to facilitate hanging gates and building penstock intakes) influencing choice of particular arch type, and the production, control, and cooling of the concrete.

Dokan dam

The Engineer (London), V. 208, No. 5415, Nov. 6, 1959, pp. 552-557

Reviewed by ARON L. MIRSKY

Brief over-all coverage of the design and construction of this dam, on the Lesser Zab River (a tributary of the

Tigris) in Iraq. The reservoir created by the dam is a multipurpose one, involving irrigation, flood control, and power generation.

Novel aspects include use of relaxation procedures, a more sophisticated method of stress analysis than usually employed, and tests on rubber models;* the construction of a high curtain of grouted rock;† and a study of reservoir operation based on a "rule curve."

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* Experimental and Mathematical Analysis of Arch Dams with Special Reference to Dokan," D. N. da G. Allen, L. Chitty, A. J. S. Pippard, and R. T. Severn; "Determination of the Stresses in an Arch Dam from a Rubber Model," L. Chitty and A. J. S. Pippard: *Proceedings*, Institution of Civil Engineers (London), V. 5, No. 3, May 1956, pp. 198-258 and 259-275, respectively; Current Reviews, ACI JOURNAL, V. 28, No. 5, Nov. 1956 (*Proceedings* V. 53), p. 557, and V. 28, No. 7, Jan. 1957 (*Proceedings* V. 53), p. 716, respectively.

†For more details, see paper reviewed above.

Design

Design of concrete tunnels (in Hungarian)

T. HAROSY, *Magyar Tud. Akad., Musz. Tud. Oszt. Kozl.*, V. 23, No. 1/2, 1958, pp. 39-62
APPLIED MECHANICS REVIEWS V. 13, No. 4, Apr. 1960

In the first part author derives an equation for the determination of the ratio of vertical and horizontal rock pressures around a circular hole, where the influence of lateral displacement and subgrade reaction is also considered. In the second part he derives a modification for the rock-pressure theory of Kommerell with the utilization of Protodiakonov's theory, based on the subsequent superposition of the compression of overlying layers resulting from the vertical displacement of a circular hole. Both his results must be regarded as mere approximations where the plastic behavior of rocks and linings is neglected, and, in addition to the defects of its principal assumptions, his results contain all those inherent to the above-mentioned original theories. Still, the way in which he deals with the problem puts some new light upon this rather intricate problem and means a step forward to its solution.

Analysis of the general hyperbolic paraboloid shell

M. N. KESHAVA RAO, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 450-454
AUTHOR'S SUMMARY

The differential equation of the membrane theory applied to a hyperbolic paraboloid (or any anticlastic shell) is of the hyperbolic type and hence the stress analysis of this shell is different from that of other shells. This difference is discussed and certain peculiarities in the boundary conditions are explained. A simple method of numerical analysis which results in fairly accurate values for stresses is given and illustrated by an example.

Design of long reinforced concrete columns

Bulletin No. 11, Reinforced Concrete Research Council of the Engineering Foundation, 1960, 70 pp. (Copies may be obtained from Portland Cement Association)

Reports research performed at the University of Illinois under sponsorship of the Reinforced Concrete Research Council. The three papers, by Bengt Broms and I. M. Viest, are reprinted from the *Proceedings*, ASCE, V. 84: ST 1, Jan. 1958; ST 3, May 1958; ST 4, July 1958; and discussions of the three papers.

Ultimate strength analysis of long hinged reinforced concrete columns

Theoretical analyses are presented for the ultimate strength of long hinged reinforced concrete columns. Both concentric and eccentric loads are considered. The treatment of concentrically loaded columns is based on Engesser's tangent modulus formula and the treatment of eccentrically loaded columns is based on the principles advanced by Kármán. Both analyses utilize the stress-strain relationship for concrete determined by Hognestad from tests of short eccentrically loaded columns.

The analyses are compared with the test results of six experimental investigations carried out in the past. The test data, including 48 concentrically and 79 eccentrically loaded long hinged

columns, and the analyses are in good agreement, thus indicating that the analyses with their basic assumptions yield reliable results.

Ultimate strength analysis of long restrained reinforced concrete columns

Theoretical analyses are presented for the ultimate strength of long restrained reinforced concrete columns. The analyses are based on the same principles as the analyses of the ultimate strength of long hinged columns presented in the paper above. The consideration of the end restraints requires the addition of a new assumption and necessitates new methods of solution.

The results of the analyses are presented in dimensionless graphs. The following variables are included: (1) concrete strength, (2) percentage of reinforcement, (3) yield point of reinforcement, (4) end restraint, (5) eccentricity, (6) slenderness ratio, and (7) duration of loading.

Design of long reinforced concrete columns

The design procedure presented in this paper involves the strength of a short column, the eccentricity determined from an elastic analysis, and a reduction coefficient. Depending on the method used in computing the short column strength, the procedure may be applied at either working or ultimate load level.

The reduction coefficient is based on the results of a theoretical analysis of the strength of long columns. It is shown that the strength of a hinged column with the eccentricity obtained from an elastic analysis represents a *lower limit* for strength of restrained columns. It is shown further that the ratio of the strength of a long column to that of a short column depends primarily on the slenderness ratio and on the ratio of end eccentricities. These two variables are taken into account in the reduction coefficient.

The design procedure is compared with the available test data for hinged columns.

Report to HRB Committee No. 1 —Parking

GEORGE W. HOWIE, *Highway Research Abstracts*, V. 30, No. 3, Mar. 1960, pp. 34-38

A report of garage design criteria studied in eight cities where a team of architects and engineers inspected 40 facilities. Discussion covers: column spacing, bay dimensions; ramp grades, radius of curvature, width; floor slope, drainage, floor finish; ventilation and fire protection; lighting; pedestrian handling; and maintenance as affected by design.

Comparison of analyses of helical stairs

V. A. MORGAN, *Concrete and Constructional Engineering* (London), V. 55, No. 3, Mar. 1960 pp. 127-132

A helical flight of stairs rises 11 ft between the basement and ground floors of a new office building at Ealing, Middlesex, and has twenty-two 6-in. risers. A landing is provided 3 ft 6 in. above the basement floor because the fire regulations do not permit a flight of more than 15 steps. Above the landing the waist of the stairway is 8 in. thick but below the landing the waist is thicker to maintain an unbroken soffit. The flight turns through 360 deg. The imposed load was assumed to be 100 psf of area in plan.

Two methods were used for analysis. In one method it was assumed that the structure was freely supported, and in the other it was assumed to be fixed at the ends. The methods are described and compared.

Slabs spanning in two directions designed by the yield-line method—Uniformly distributed loads

G. M. MILLS, *Concrete and Constructional Engineering* (London), V. 55, No. 3, Mar. 1960, pp. 111-119

Rectangular panels of slabs spanning in two directions may, according to British Standard Code of Practice No 114, be designed by three methods, one of which requires the moment of resistance to be computed by the load-

factor method and the determination of the strength by Johansen's yield-line theory. This method may result in considerable saving of material and gives greater flexibility in design than alternative empirical methods.

Design formulas, a nomogram for bending moment coefficients, and a graph for correction of corner effects is included, as well as an example problem.

Proposed classification of shell roofs

JAL N. BHARUCHA, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 419-421
AUTHOR'S SUMMARY

A great variety of geometrical forms are now being used by the shell designer and some system of classifying them has therefore become necessary. An attempt is made in this paper to set out a pattern of classification to cover presently known forms of shell roofs.

Computer program for multicylindrical shell roofs

J. E. GIBSON, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 407-410
AUTHOR'S SUMMARY

Describes how shell roof calculations have been programmed for the Ferranti "Mercury" electronic digital computer and then gives a typical set of results. A simple shell without edge beams is considered in some detail and the course of the program may be followed by means of the flow diagram. More complicated programs are then described dealing with calculations for multishells.

The only information needed to be presented after the program tape has been fed into the computer are the basic dimensions of the cylindrical shell to be designed. Thus the length, radius, thickness, and half angle of the shell, together with the load per unit area and the relevant weights and dimensions of the edge beams with prestressing forces and cable eccentricities, if required, are punched on a data tape and subsequently fed into the computer. The program is then automatic and the output produces the required shell and edge beam forces and moments in tabular form.

The time required to produce such results for a single shell with edge beams, allowing unrestricted movement of the edge beams, is 25 sec; the same calculation on a desk calculating machine would require some 18 hr.

Analysis of domical shells — Modifications to general equations

A. CARBONE, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 438-444
AUTHOR'S SUMMARY

Examines the general equations of deformation suggested by some authorities, and points out that some of them do not agree with the definitions of some of the elements which characterize the deformations. The paper also indicates how these definitions should be modified.

The necessity of introducing a correction term which appears in two of the equations of elasticity is also discussed. The particular case of the hollow sphere subjected to internal pressure is considered, and it is shown that the bending moment calculated by some authors does not exist. This is also confirmed by applying a mathematical solution to this case.

Analysis of northlight shells

G. S. RAMASWAMY and M. RAMAIAH, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Supplement, Dec. 1959, pp. 1-3
AUTHORS' SUMMARY

There is at present no accepted method for the design of northlight shells. The analysis of northlight shells is much more difficult than that of cylindrical shells on account of the lack of symmetry and the indeterminate boundary conditions that prevail at the top and bottom edges of the shell. A correct formulation of the problem would involve eight boundary conditions, four at each edge, and eight arbitrary constants. A few simplifying assumptions are, therefore, usually made in the analysis of such shells. The paper compares such approximate approaches as the Tottenham method and Lundgren's

beam method with the exact method to assess the degree of approximation involved. The objects of the study are to determine if the approximate methods err on the side of safety and whether the additional labor involved in using the exact approach is justified.

Discontinuity stresses in beams on elastic foundations

JOSEPH PENZIEN, *Proceeding*, ASCE, V. 86, ST 4, Apr. 1960, pp. 67-93

AUTHOR'S SUMMARY

A method of analysis is presented for obtaining discontinuity stresses in beams on elastic foundation and cylindrical shells subjected to axisymmetrical loading. Also presented are the solutions of numerous basic cases which provide the necessary numerical constants required in the proposed method of analysis. Several examples are included.

Concrete formwork designer's handbook

H. R. GILL, Concrete Publications, Ltd., London, 1960, 160 pp., $3.50

This book is concerned only with the structural design of formwork as applied to timber and steel. The various factors affecting the lateral pressure of freshly placed concrete against the forms are evaluated and the stresses permissible in timber, plywood, and steel under various conditions are given. The design aids are supported by the basic formulas from which they are derived; derivations of the formulas are given in an appendix. The book gives more than 50 nomograms, tables, and other charts for the rapid design of the components of the formwork for vertical or inclined walls, straight or curved walls, beam and slab construction, dams, and continuously moving forms.

Despite the need for greater accuracy than is frequently used in the design of formwork, because of the uncertain values of the variables and the temporary nature of the structure, it is not practical to devote as much time and care to such designs as is normally given to the design of permanent structures. However, if some approximations are made a series of simple formulas can be derived to give the sizes required and nomograms can be constructed to simplify the work. The latter formulas and nomograms occupy a large part of this book and are readily adaptable for most types of form construction.

Dynamic effects of earthquakes

RAY W. CLOUGH, *Proceedings*, ASCE, V. 86, ST 4, Apr. 1960, pp. 49-65

AUTHOR'S SUMMARY

The principal factors controlling the dynamic response of structures to earthquakes are summarized, and are related to the lateral force provisions recommended for inclusion in the Uniform Building Code by the Structural Engineers Association of California. These provisions are seen to conform well with the concepts of dynamic theory.

Structures of high buildings (Estructuras de edificios elevados)

JULIO RICALDONI, Facultad de Ingenieria y Agrimensura, Montevideo, Uruguay, No. 18, Dec. 1958, 46 pp.

Reviewed by CELSO A. CARBONELL

An interesting study in the problem of stability of high buildings, considering wind forces and seismic movements. Presents a discussion of the factors that influence the selection of the most convenient structure and also a method for calculation.

Simplified design of reinforced concrete

HARRY PARKER, John Wiley & Sons, Inc., New York, 2nd Edition, 1960, 303 pp., $6.50

The second edition of this text has been prepared in compliance with the recommended unit stresses and design procedure and requirements as given in the ACI Building Code (ACI 318-56). As with the first edition, the intent is that the book be used not only as a textbook but also for home study. Simple concise explanations of the design

of the most common structural elements have been given rather than discussions of the more involved problems. A new chapter on prestressed concrete has been added; its purpose is to explain the theory of prestressing. An illustrative example shows—by the design of a conventional reinforced concrete beam, and the same beam designed as prestressed concrete—why prestressing is advantageous in the economical use of materials.

The usual tables necessary in the design of reinforced concrete are included and also where practicable, safe load tables have been added.

Materials

Mechanical classifier of aggregate according to strength (in Russian)

B. I. KURDENKOV and A. P. FILATOV, *Avtom Dorogi* (Moscow), V. 21, No. 12, 1958, pp. 27-28; *Road Abstracts*, V. 26, No. 3, Mar. 1959, pp. 55-56

HIGHWAY RESEARCH ABSTRACTS
Oct. 1959

A description is given of the portable plant designed by N. K. Timchenko to separate off weaker material from aggregate of a given strength. The material is carried from a hopper by a vibrating belt and is fed through a funnel with an adjustable angle of feed on to an electrically driven drum revolving at 203 rpm. The stronger, more resilient material rebounds from the drum and is collected separately from the less resilient weaker material which is carried forward by the movement of the drum to the reject funnel. Recommendations are given for the angle of feed required for a given aggregate strength. The efficiency of the plant is impaired if there is a large proportion of weak material or more than 6 percent moisture content. The results are discussed of controlled experiments. carried out by NIIZh (Concrete Research Institute, USSR). Aggregate strength was checked by the barrel attrition test and the manual shear test devised by Timchenko in which the strength of specimens is measured by an instrument resembling a pair of cutting pliers; a dial records the pressure required to cause shear of fracture of the specimen. The results of successful tests carried out by the device, are recorded, and an illustration is given of the apparatus in operation.

Shrinkage during the setting, above and below the freezing point, of portland cement to which chlorides have been added (in Russian)

V. V. NEKRASOV and G. A. SHISHO, *Journal of Applied Chemistry* (Moscow), V. 31, 1958, p. 1460; *Transactions*, British Ceramic Society, V. 58, No. 4, Apr. 1959, p. 112A

HIGHWAY RESEARCH ABSTRACTS
Oct. 1959

When portland cement sets in the presence of up to 10-11 percent $CaCl_2$, the water-cement ratio being approximately 0.5, the hydration below 0 deg increases, in contrast to decreased hydration that occurs below 0 deg in the absence of chlorides. This is attributed to the formation of complex Ca chlorosilicates. At temperatures above 0 deg these components are less stable and the density and strength of concretes containing these chlorides may decrease at higher temperatures. Reactions between Ca aluminates and $CaCl_2$ below 0 deg have less influence on the hydration of portland cement.

Gravimetric determination of strontium oxide in portland cement

C. L. FORD, ASTM *Bulletin*, No. 245, Apr. 1960, pp. 71-75

AUTHOR'S SUMMARY

Determinations of the strontium oxide content of portland cements make possible more accurate calculations of the potential tricalcium silicate contents thereof. The study of four methods reported for gravimetric analysis for strontium was made to find one that could be used or modified to give good results in the determination of the small amounts of strontium present in cement. The method selected is based on the extraction of calcium nitrate with 80 percent nitric acid from the dry mixed nitrates of calcium and strontium, and the quantitative determination of the residual strontium ni-

trate. The accuracy of the procedure was demonstrated by analyzing high calcium samples to which known small amounts of strontium had been added. The method used is described briefly in the body of the report and a more detailed procedure is presented in the Appendix.

Pavements

London to Birmingham motorway

The Engineer (London), V. 208; No. 5415, Nov. 6, 1959, pp. 544-548; No. 5416, Nov. 13, 1959, pp. 586-591

Reviewed by ARON L. MIRSKY

Description (with excellent photographic coverage, especially on pp. 547, 548, 586, 588) of England's first* real expressway,† designated the M1. Its over-all length of 75 miles is provided with dual three-lane roadways for most of its length, with dual two-lane roadways used for the spur roads at either end. It is divided into a 55 mile main section, the 12 mile St. Albans bypass, and the short Dunchurch bypass.

*Excepting the 8-mile Preston bypass.
†And, as might have been expected, it contained "bugs" (See, for example, *Engineering News-Record*, Dec. 17, 1959, p. 126).

How a dual-drum paver operates

Public Roads, V. 31, No. 1, Apr. 1960, p. 11

Sketches and text describe and illustrate the functioning of a dual-drum paver—the machine generally used on site to mix portland cement concrete for highway pavement—by following a single batch of cement and aggregate completely through the paver.

Study of 34-E dual-drum pavers

DONALD O. WOOLF, *Public Roads*, V. 31, No. 1, Apr. 1960, pp. 1-10

AUTHOR'S SUMMARY

Summarizes data obtained in 16 studies of the mixing of portland cement concrete in 34-E dual-drum pavers, conducted by 13 state highway departments. Determinations were made of the desirable time of mixing and the amount of overload which could be permitted. The results indicated that the highest strengths were attained at a 60-sec mixing time, exclusive of transfer time, and that an overload of 10 or 20 percent could be tolerated. It was also found that little loss in strength of the concrete resulted when the mixing time was reduced to 40 sec, exclusive of transfer time, but when the mixing time was reduced to 30 sec a loss in strength of 5 to 6 percent occurred. A mixing time of more than 60 sec, exclusive of transfer time, was found to be undesirable since it involved waste of effort and resulted in loss of strength without gain in the uniformity of the concrete.

Precast Concrete

Precast concrete components hold factory cost to $6.68 per sq ft

Building Construction Illustrated, V. 28, No. 2, Aug. 1959, pp. 34-35

The low construction cost of this factory was largely possible through use of tilt-up panels for walls and precast prestressed concrete channel joists and slabs for the roof. Several cross-sectional drawings are included.

Garage specifications: Beauty and economy

ALFRED ZWEIG, *Engineering News-Record*, V. 164, No. 13, Mar. 31, 1960, pp. 46-48

Describes the design and construction of the parking garage built for the Henry Ford Hospital, Detroit. Unique feature of the garage was the exterior of hyperbolic paraboloid precast wall panels of quartz aggregate and white cement used to make the structure architecturally acceptable in a residential neighborhood.

Precast concrete web forms unique dome

Contractors and Engineers, V. 57, No. 5, May 1960, pp. 80-83

Building three open space frames for a conservatory in the shape of a prolate spheroid gets complicated! Each dome, 70 ft high and 140 ft in diameter, will be built of six different sizes of precast units. There are 275 pieces of precast concrete that must fit together

exactly to make the space frame. The units have three basic shapes: hexagonal, triangular, and diamond-shaped. The elements were cast in concrete forms.

A system of supporting pipes and I-beam stringers was employed to support the units during erection. The I-beams outlined roughly the shape of the dome; adjustable steel chairs, welded to the I-beams, outlined it precisely.

Autoclaving in the U.S. — A progress report

CARL A. MENZEL, *Concrete Products*, V. 63, No. 5, May 1960, pp. 24-31

AUTHOR'S SUMMARY

A review of autoclaving in the United States. Methods of single and two-stage autoclaving are explained and their effects are explored; some attention is also given to carbonation. The author traces the rise of autoclaving and relates it to block industry growth.

Concrete roof tiles

Concrete Products, V. 63, No. 4, Apr. 1960, pp. 40-41, 44-45, 63

A comparison of production methods in the United States and in Europe.

Concrete railway sleepers

F. S. HESPE, *Constructional Review* (Sydney), V. 33, No. 1, Jan. 1960, pp. 30-38

Describes the four types of concrete railway ties available in Australia. (Concrete ties have not been used in any quantity in Australia although the government has used some in experimental sections of track. Some private companies are using them in their own sidings.)

New materials and method of construction (in French)

Cahiers No. 41, Centre Scientifique et Technique du Batiment, Paris, Dec. 1959

Reviewed by A. W. COUTHS

This particular report of the French Scientific and Technical Center of Building Construction should be of particular interest to those engaged in precast and prefabricated concrete construction. Graphic and verbal descriptions of 27 different patented methods of prefabricated floor and wall elements are presented in a highly lucid manner which should prove stimulating to American specialists in this field. Elements can be constructed by one of the three alternative methods: (1) pretensioned prestressed concrete, (2) pretensioned prestressed and reinforced concrete, (3) reinforced concrete.

While French engineers have been slow toward adapting precast techniques, this report indicates that the trend is reversing and should provide readers with good practical ideas on improving their own methods.

Properties of Concrete

Does calcium chloride attack steel? (Ist Calciumchlorid stahlaggressiv?)

W. LIEBER and K. BLEHER, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 6, June 1959

AUTHORS' SUMMARY

Calcium chloride and alumina are likely to develop Friedel salt if conditions are favorable, for instance, when mixing cement containing calcium chloride. At the same time, the double salt $3CaO \cdot CaCl_2 \cdot 15H_2O$ may result from calcium hydroxide and calcium chloride. These two compounds are non-hygroscopic. Tests carried out by several authors reveal that corrosion is avoided if reinforcing steel is tightly bedded in moderately lean concrete with low water-cement ratio. However, in the case of lean and improperly compacted concrete with a relatively high water-cement ratio, corrosion may occur, whether the cement contains calcium chloride or not.

Introduction to the technological theory of the shrinkage and swelling of concrete (in Russian)

VISHNEVETSKII G. D., 15th Scientific Conference, Leningrad Engineering-Construction Institute, Leningrad, 1957, pp. 356-363

APPLIED MECHANICS REVIEWS Oct. 1959

It is assumed that the concrete body is homogeneous in regard to the processes of hydration, diffusion, humidity, and thermal conductivity, isotropic and

quasi-homogenized, porously solid. The physico-chemical state of the concrete is described with the aid of four parameters: temperature, extent of firmly-bound water, extent of weakly-bound water ,and extent of free water. The last three parameters are expressed as "parts" in the water-cement relationship. The shrinkage is expressed as the linear ratio of the difference between the initial and final water-cement relations. The expressions for the calculated values are recorded in a general form. The equations contain many coefficients but the methods for their determination are not disclosed.

New method for accelerating the hardening of concrete (in Hungarian)

Gy. Balazs, J. Kelemen, and J. Kilian, *Epitöanyag* (Budapest), V. 10, No. 9, 1958, pp. 326-331

Hungarian Technical Abstracts V. 11, No. 3, 1959

In view of the widespread use of prefabrication the accelerated hardening and higher strength of concrete assumes increased significance. Methods used for this purpose, e. g., steam curing, use of chemicals, special cements, and the wet grinding of cement pastes are limited in their applicability partly by the specified quality and partly by economical considerations. A new method, first described by Duriez, has therefore been tested.

This method is based on nuclei of crystallization which are the hydration products of cements under various conditions. Best results have in general been obtained by adding such nuclei in a quantity of 2 percent by weight of cement. An increase in strength of 10 to 30 percent could be observed by the second and seventh days respectively. Acceleration of hardening by the addition of nuclei of crystallization can be combined with other more conventional methods. Nuclei of crystallization used together with $CaCl_2$ or with $CaCl_2$ and bentonite resulted in increasing the strength by 40 to 45 percent when 2 days old.

Following a short description of the hardening process, experimental results obtained with three different cements of "C 500" grade and with one slag cement of "C 400" grade, are described. During these experiments nuclei of crystallization produced in 100 C steam as well as in a damp atmosphere of 90 to 100 percent relative humidity at 20 C have been applied.

Internal stresses in concrete

Knud E. C. Nielsen, RILEM *Bulletin* (Paris), No. 1, New Series, Mar. 1959, pp. 11-20

Describes preliminary tests to measure internal stresses—aggregate stresses and strains—in unloaded concrete specimens cured in air or in water followed by air curing. This was done by replacing some of the aggregate with "artificial stones" (internal stress gages) and measuring the strain on them. The gages are designed to approximate, as clearly as possible, the shape, size, and rheological properties of stone, with a maximum size of 40 mm. Ten different types of gages were used in the tests.

Rapid estimation of cement content in mortar and concrete

R. L. Dewan, *Indian Concrete Journal* (Bombay), V. 33, No. 4, 1959, pp. 132-133

Road Abstracts Sept. 1959

The method described consists essentially of determining the calcium content of mortar and concrete. It is said to have advantages over the conductometric and adsorption methods since it is influenced neither by the conductivity of the salt content of water used in the mix nor by the adsorption properties of any soil present. One g of finely powdered mortar is digested with 10 to 15 ml of hydrochloric acid solution for 3 to 5 min on a heater. The solution is filtered, washed, and made up to 100 ml, half of which is then just neutralized with liquid ammonia. Just enough glacial acetic acid is added to dissolve the precipitates of Al_2O_3, Fe_2O_3, etc., formed. By adding ammonium oxalate in excess to this solution, a copious granular precipitate of calcium oxalate is obtained. The precipitate is

filtered, washed, and dissolved in a sulphuric acid solution and then titrated with standard potassium permanganate solution to determine the percentage of calcium. A graph can be drawn up showing the percentage of calcium against percentage of cement in different sand-cement mixes. The percentage of calcium in different types of cement does not vary widely, but more accurate estimates of cement content are obtained in separate graphs which are prepared for each type of cement used.

General

Soils and soil engineering

R. H. KAROL, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1960, 194 pp., $11.65

This book will make a good reference for the practicing engineer and soil scientist and will be of value to educators as well as technological authorities in government and private industry. Using a scientific approach the book provides undergraduate students with a working knowledge of soil mechanics.

The first portion of this book is devoted to a discussion of soils and their physical properties. Insofar as is possible, limits of properties and characteristics are given for various soil types, to furnish a basis upon which further experience can be built. The data presented for this purpose should be considered as qualitative and should not be used for direct application in soil analyses.

The latter sections deal with the methods and techniques of applying the principles of statics to the solution of typical soil engineering problems in slope stability, bearing capacity, retaining structures, and pile foundations. The scope of this portion of the book is of necessity limited to the more usual problems in these fields.

The author describes testing methods and machines, thus indicating to the reader how the necessary soil data is obtained. Since the author is considered to be an authority on soil stabilization and foundation design, the book includes basic original material on soil stabilization by chemical grouting which is now receiving attention by soils engineers throughout the world.

Concrete — The vision of a new architecture

PETER COLLINS, Horizon Press, Inc., New York, 1959, 308 pp., $12.50

In almost amazing detail the vicissitudes of the development and acceptance of concrete and reinforced concrete in Europe and America are traced in an entertaining and instructive style. The account of the efforts of promoters, contractors, engineers, and architects to overcome false ideas of using the cheapest of materials, lack of skill of workmen, inflexible and unyielding building regulations and officials, and the excesses of their own fraternity is engrossing reading. The reader rubs elbows with such greats as Coignet, Ward, Hyatt, Ransome, Hennebique, and finally the architect Perret. Worthwhile reading for anyone interested in the technical and architectural developments in concrete in the nineteenth and twentieth century.

American building art

CARL W. CONDIT, Oxford University Press, New York, 1960, 372 pp., $12.50

A history of American activity in the building field during the nineteenth century industrial revolution. Interesting information is given on the evolution of true framing systems and development of the materials which made them possible. Major subjects in this over-all survey of the building field are wood framing, iron framing, wood bridge trusses, iron bridge trusses, suspension bridges, iron arch bridges, railroad trainsheds, and concrete construction. Because of the broader scope of the book, its detail and discussion of concrete construction is much less complete than that found in *Concrete — The Vision of a New Architecture* by Peter Collins (see above).

ACI NEWS LETTER

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West to Tucson

13th Regional Meeting

The ACI 13th Regional Meeting will be held under the sunny skies of Tucson in friendly Arizona, October 31 to November 2 at the Pioneer Hotel.

The 3-day program will be highlighted by joint ACI-ASTM technical sessions. Monday, October 31, is designated committee day with technical sessions convening the following two days. The program on Tuesday, November 1, will include a morning session on construction and an afternoon session on research in design.

Wednesday, November 2, will feature concurrent technical sessions in the morning: ACI design session and ASTM research session. The sessions will be significantly different so that the ASTM session will appeal to those from both groups interested in materials. The joint afternoon session will be comprised of papers which should be of general interest to ASTM and ACI members.

A joint ACI-ASTM luncheon on the day of the concurrent sessions has been arranged with A. Allan Bates, vice-president, Research and Development, Portland Cement Association, Chicago, newly elected president of ASTM as guest speaker.

Gene M. Nordby, head, civil engineering department, University of Arizona, is general chairman of the meeting with Andrew Ross, vice-chairman and Joseph Toth, treasurer,

both of the University of Arizona. Edward Mangotich, Portland Cement Association, is on the executive planning committee.

Others assisting in arranging the meeting are: Billy J. Shell, San Xavier Rock and Sand Co., technical program chairman; Lee Lawrence, civil engineer, finance chairman; Edward Mangotich, PCA, in charge of exhibits; Gene Anderson, civil engineer, publicity chairman; and Andrew Marum, consulting engineer, membership chairman.

Maurice LeGaard is in charge of the ladies' program; James Kreigh, educational contacts; and K. K. Kienow, registration. The latter three members of the local planning committee are all affiliated with the University of Arizona.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete

William W. Karl
Lehigh Materials Co.
Tamaqua, Pa.

Ivan L. Lynn
Ideal Cement Co.
Fort Collins, Colo.

Committee 339, Allowable Stresses in Reinforcement

A. J. Mullkoff
Concrete Steel Corp.
Detroit, Mich.

Committee 401, Specifications for Structural Concrete

Russell S. Fling
Russell S. Fling and Associates
Columbus, Ohio

Committee 402, Concrete Floor Finishes

James E. Backstrom
Bureau of Reclamation
Denver, Colo.

Committee 505, Design and Construction of Reinforced Concrete Chimneys

T. R. Galloway
Consolidated Edison Co. of New York, Inc.
New York, N.Y.

L. A. Irvin
Bechtel Corp.
San Francisco, Calif.

Committee 616, Coatings for Concrete

J. E. Ilaria
Lock Joint Pipe Co.
Wharton, N.J.

Committee 712, Precast Structural Concrete Design and Construction

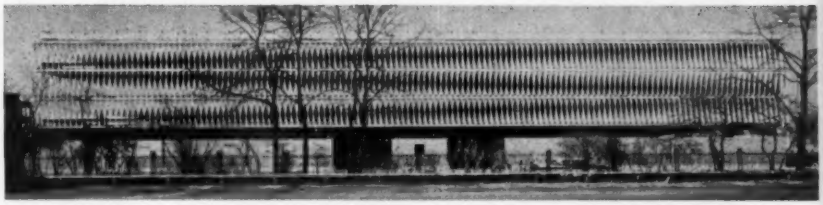
J. L. Peterson
J. L. Peterson, Inc.
Long Beach, Calif.

Modular standards work to resume

Dimensional standards for all major categories of building materials will soon be placed under intensive study by appointed representatives of associations and societies involved in building industry activities.

The occasion is a forthcoming meeting of Sectional Committee A62 of the American Standards Association on the Coordination of Dimensions of Building Materials and Equipment. Secretarial activities for the committee are a function of the recently formed Modular Building Standards Association, a nonprofit organization initiated and sponsored by the American Institute of Architects, Producers' Council, National Association of Home Builders, and the Associated General Contractors of America for technical educational and standards activities connected with the furtherance of modular practices in the construction industry.

Precast panels



Dress up parking garage

■ To BUILD A PARKING GARAGE that would fit into a residential neighborhood and blend with the hospital it was to serve, Albert Kahn Associated Architects and Engineers, Inc., Detroit, made interesting use of "twisted" precast concrete panels. The panels are hyperbolic paraboloids 24 in. wide at base and top, 18 in. at the waist with a 90 deg twist between top and base. They are 7 ft 4½ in. high and vary in thickness from 2½ to 3½ in.

Cast in "right-hand" and "left-hand" twists—the rows are alternated top to bottom and between adjacent sides—the panels present an everchanging face to the viewer. The eye sees no two panels alike from any vantage point; and each panel is viewed differently from each vantage point. The white cement panels mask the irregular lines of the parked cars on the second, third, and fourth floors.

The garage serves the staff of the Henry Ford Hospital, Detroit, parking 870 cars on four covered floors and the roof. The 288,840 sq ft of floor space is divided almost equally between two "split-level" sections. The sections are about 118 x 214 ft and are offset from one another by a half story vertically and one half column spacing horizontally (see cover photo). The two sections are connected by four ramps, two in each direction. The core area between the two sections houses the elevators and stairwells, and the unused portion serves as a natural light "corridor" from ground to roof. On the first floor, one corner of the structure (5600 sq ft) is enclosed to house servicing and car washing facilities.

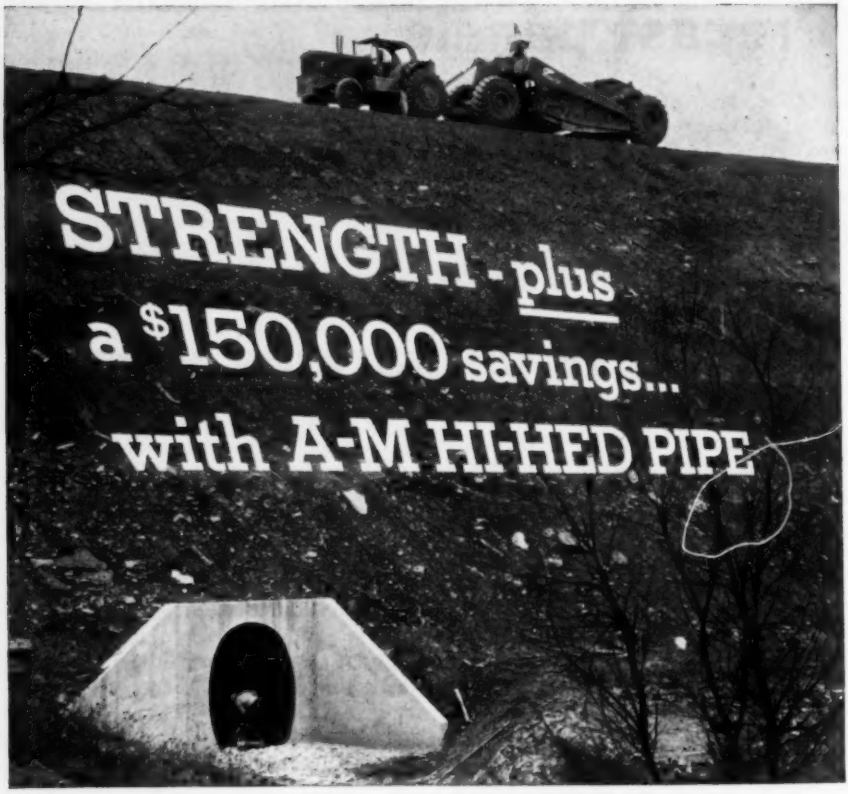
Structural frame

The garage is of flat slab construction. The 11 in. slab is set on rectangular columns, 18 x 36 in. in cross section, with 5 in. drop panels. The large column cross section and the column capitals contribute to the rigidity and economy of the structure. The large column dimension is always in the direction of traffic and does not limit the parking space.

Columns are spaced at 27 ft 1½ in. in the east-west direction, and alternate between 28 ft and 32 ft in the north-south direction. The slab cantilevers 10 ft in the east-west direction and 14 ft north and south.

Design story height is 8 ft 6 in., with the first floor in the taller portion one-half story higher. The first floor is walled with brick, to match the existing hospital, backed up with concrete block. The brick walls in both sections are the same height, emphasizing the difference in height of the two "levels." The elevator penthouse is also walled with the red brick.

The floors of the structure are treated with crack sealer to prevent salt,



STRENGTH was the first requirement for a 4,000 foot drainage structure under the new east-west runway at Greater Pittsburgh Airport. Precast concrete Hi-Hed pipe provided this essential strength (50% stronger on the average than equivalent size round pipe), plus a big bonus in the form of an estimated saving of $150,000—approximately 18% savings over competitive material.

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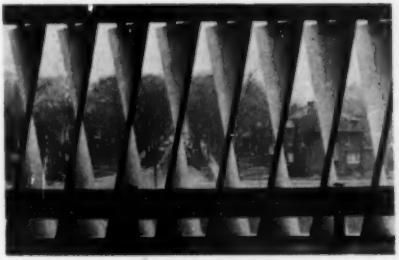
carried in by the cars during the winter, from penetrating the concrete and rusting the reinforcement. The sealer used is expected to give protection for about 5 years before being replaced. To help assure continuous removal of water from the slabs of the open structure, even in winter months, thermostatically controlled electric heaters were installed at the bends in drainage conductors.

Precasting panels

The precast facing panels presented the contractor with some forming problems. The original plan called for casting them against concrete forms. The weight of this type of form made their use prohibitive. Experimentation also showed that laminated steel forms would produce problems of handling. The solution was to use Fiberglas formed to the required shape and mounted on steel angles. The forms were made in two parts each reinforced with a framework of steel pipe. The panels were cast through one edge of the joined form and vibrated externally on a low, flat vibrating table. There were 15 right and 14 left forms required for the job.

Each panel was reinforced with three #3 bars vertically and eight #3 bars horizontally. Only straight bars were required because of the hyperbolic paraboloidal shape of the panels. The concrete used was proportioned for 5000 psi compressive strength at 28 days. The mix used a milky-white quartz aggregate, fine Ottawa sand, and white cement. Steel inserts were welded to the reinforcing steel on the inside face to be used in securing the panels after erection.

After placing and vibrating, the panels were kept in the forms for 24 hr. After removal from the forms, they were moist cured for 7 days in a curing chamber constructed for the purpose where they were sprayed from "soaker" type garden hose in the ceiling. Extreme care was required to keep panels clean during handling and curing.



View from inside of garage showing panel connections

The 445-lb panels were cast in the city but not at the site of construction. They were transported to the site on a 5-ton stake body truck, 25 panels per load.

Erecting panels

The panels were hoisted to their final position and placed in keyways provided in the lower slab. Above, a continuous insert had been cast in the beam to which a galvanized clip angle slotted both vertically and horizontally, secured the top of each panel. The horizontal slot in the clip angle aided in positioning the panels in the continuous insert. The vertical slot allows for movement of the cantilevered slab without transmitting load to the panels; the slightest movement against the panels could cause unsightly damage. The slabs, however, proved to be quite rigid, deflecting only about ⅛ in. Once the panels were positioned, the clip angles were welded to the continuous insert in the upper spandrel beam and the keyway at the bottom of the panel grouted.

In all, there were 1716 panels used. The cost was under $50 per panel of which less than 15 percent was form cost. The over-all cost of the panels was about that of a conventional brick curtain wall. Total cost of the building was about $1225 per parked car.

Alternate framing system

An alternate to the flat slab method was proposed by the engineer and bids taken on prestressed lift-slab construc-

tion with steel columns. This was the first time bids had been taken on this type of construction in the Detroit area. The alternate proved to be 13 percent higher than the system used. The increase due to using lift slab construction was insignificant to that caused by the prestressed concrete proposal even though the design would not have provided sufficient prestress to provide a crack-free floor.

Credits

Built for the Henry Ford Hospital, the parking garage was designed by Albert Kahn Associated Architects and Engineers, Inc., Detroit. Alfred Zweig was project engineer and in charge of the structural design. Contractor was Darin and Armstrong, Inc., Detroit, and the panels were precast and erected by Truscon Laboratories, Detroit.

LOOKING AHEAD

Aug. 15-26, 1960 — Summer Institute on Nondestructive Testing, Sacramento State College, Sacramento, Calif.

Aug. 29-Sept. 2, 1960—Symposium on Reinforced Concrete Design, Queen's University, Kingston, Ont., Canada.

Sept. 27-30, 1960 — 6th Annual Convention, Prestressed Concrete Institute, Statler-Hilton Hotel, New York, N.Y.

Oct. 10-13, 1960—American Mining Congress, Metal Mining and Industrial Minerals Convention and Exposition, Las Vegas, Nev.

Oct. 13-15, 1960—75th Anniversary and National Convention, Tau Beta Pi, Lehigh University, Bethlehem, Pa.

Oct. 31-Nov. 1-2, 1960—13th Regional Meeting, American Concrete Institute, Pioneer Hotel, Tucson, Ariz.

Nov. 3-4, 1960—Annual Meeting, National Slag Association, Hotel Mayflower, Washington, D.C.

Nov. 14-19, 1960 — Public Works and Municipal Services Congress and Exhibition, Olympia, London, England

Dec. 12-15, 1960—First Industrial Building Exposition and Congress, New York Coliseum, New York

Jan. 5-7, 1961 — FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

Jan. 9-11, 1961—Annual Convention, Concrete Products Manufacturers Association of Quebec, Chateau Frontenac, Quebec, Canada

Jan. 17-19, 1961—44th Annual Convention, National Crushed Stone Association, Hotel Americana, Bal Harbour, Fla.

Jan. 23-26, 1961—31st Annual Convention, National Ready Mixed Concrete Association, Hotel Americana, Bal Harbour, Fla.

Jan. 30-Feb. 2, 1961—12th Biennial Concrete Industries Exposition, National Concrete Masonry Association, Cobo Hall, Detroit, Mich.

structure or foundation troubles?

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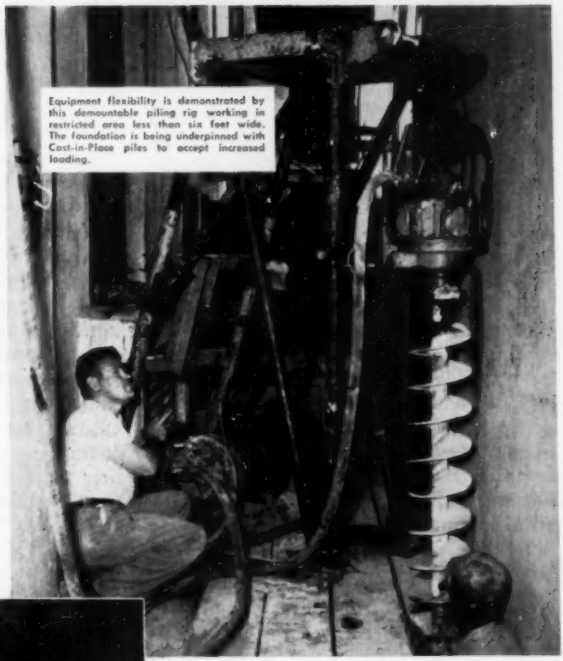

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Dome Forms Carried Up by Slip-Form for 2,950,000-Gal. Prestressed Concrete Standpipe

As part of a $1,200,000 water expansion program, Sayreville, N. J., recently completed a 2,950,000-gal. prestressed concrete standpipe. The cylindrical tank is 122 ft high with a 65 ft inside diameter. The tank was built by the slip-form method and the forms for the dome were carried up with the wall forms.

Foundation

The foundation of the standpipe was comprised of a reinforced concrete footing ring 2 ft thick x 8 ft wide and a 1 ft thick foundation slab inside the ring flush with the top of the footing. A continuous plastic waterstop was cast in the footing and extended horizontally into the foundation slab. A second waterstop extended vertically upward into the standpipe. Mix proportions for the foundation were: 550 lb cement, 1329 lb sand, 1900 lb 1-in. stone, and 40 gal. of water.

Two sections of 12-in. cast-iron pipe were installed in the earth before placing the footing and slab. These sections were enclosed in lean concrete and extended horizontally under the footing and vertically up through the floor slab.

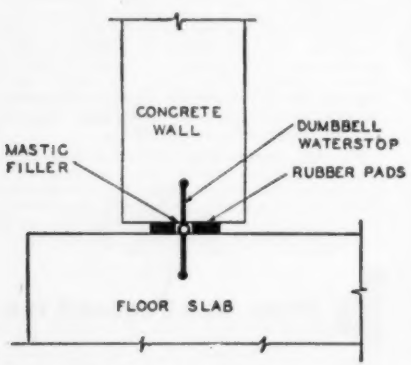


Fig. 1—Base pads and waterstop between tank wall and floor slab

Base pads

The wall of the standpipe was supported on rubber pads, one on either side of the waterstop (Fig. 1). With this method of support, when prestressing wire is wrapped around the circumference of the tank, the base of the wall is compressed uniformally, and is not subjected to nonuniform stresses which result when the wall rests directly on the foundation. Under such conditions, even when lubrication is applied between wall and foundation, differences in friction from one location to another can result in nonuniform stresses which can cause cracks and leaks.

The pads also avoid large bending moments in the wall since the base of the wall is not rigidly fixed. The rubber pads begin to deform with the first application of the prestressing wire and the deformation gradually increases as prestressing progresses. Vertical stresses are reduced to approximately 30 percent of those which develop with the lubricated sliding base. Thus, the thickness of the wall can be less than with a sliding base and is usually determined by circumferential prestressing rather than by the vertical moments.

Wall

The wall was constructed by the slip-form method (Fig. 2), 45 in. high wall forms being hydraulically raised approximately 1 in. every 4 min as placed concrete hardened. Placing the concrete was continuous and it took 117 hr to complete the wall concreting, two 12-hr shifts being employed.

Mix proportions for the wall concrete were: 637 lb cement, 1253 lb sand, 1900 lb 1-in. stone, and 40 gal of water. The slump varied from 3 to 5 in.; a 3-in. slump was used during the cool of the evening and 5-in. slump was used during the hot hours of the day. No admixture was used.

The slip forms (Fig. 3) were suspended from yokes which straddled the wall. Legs of the yokes extended downward along the outside of the forms and supported them on shoes which fitted under wales of the forms. The yokes had hydraulic jacks built into their midsections and elevated the forms by climbing jack-rods cast in the wall as it was constructed.

Wall reinforcement consisted of ½-in. vertical bars placed toward inner and outer wall surfaces at 12-in. intervals, and horizontal bars tied at 12 in. intervals to the vertical bars. These bars were installed as the forms raised. Vertical reinforcing bars were held in place by jigs on the forms.

Ready-mixed concrete was trucked to the site, emptied into a 1-cu yd bucket and hoisted to a hopper on the working platform (Fig. 4). Buggies carried the concrete to the forms and it was then worked into place and consolidated by vibrators. Inner and outer surfaces of the wall were hand finished from scaffolds hung from the forms.

Fig. 2 — Slip-forms about halfway up standpipe

When the wall was completed, ⅝-in. bars 18 in. long, pointed at one end, were driven through the forms and into the concrete just below the upper wales to take the load off the forms. Yokes, hydraulic oil lines, and the pump were then dismantled and the platform cleared..

A ⅝ in. thick rubber pad was cemented to the top of the tank wall, separating it from the dome ring and

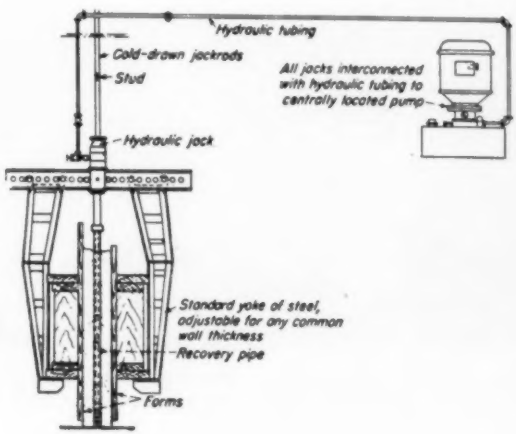


Fig. 3 — Slip-form arrangement for raising the wall forms

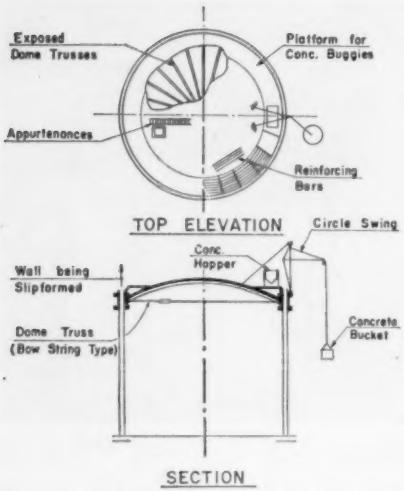


Fig. 4 — Schematic showing how the dome forms were lifted with the wall forms, as well as the positions of the hoist and hopper on the working platform

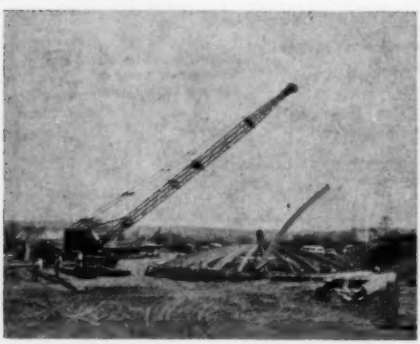
again allowing wall and dome to deform independently.

Dome

The form for the roof dome was built on the ground (Fig. 5 and 6) and carried up with the wall slip-forms (Fig. 4).

Twenty-four arched trusses for the dome form radiated outward from the center of the structure to the slip-forms. Twenty of the trusses were of the bow-string type with tie rods at the base whereas four were regular trusses of wood strut construction. The four wood strut trusses were arranged to form four 90-deg quadrants and five bow-string trusses were equi-spaced within each quadrant. The tie rods of the bow-string trusses terminated at holes in a 36-in. diameter steel plate at the center of the structure. The arches of these trusses terminated at the center at wood hypotenuses of the 90-deg angles made by the four main trusses. The bow-strings were bolted to the arches at the slip-form ends. The trusses rested on the wales of the wall forms, the dome sheathing being sufficient to keep them erect, and the roof form was carried up with the wall forms.

The dome was reinforced with 4 x 4 in. x #6 x #6 wire mesh and by seven circumferential rings of #3 bars tied to #3 radial bars spaced 8 in. on centers at the edge of the dome. The underside of the dome was in the form of a segment of a sphere having a radius of 69 ft ¾ in. The dome tapered in thickness from 2½ in. at the center to 4¾ in. at the outer edge, so the outer surface does not exactly follow the spherical contour. The dome concrete was applied in the form of pneumatic



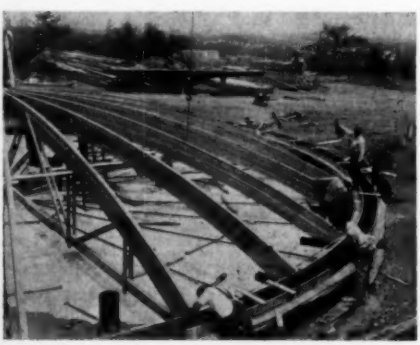


Fig. 5 (left)—Trusses for dome formwork being placed in position. Fig. 6 (right)—Wood trusses for dome form and wall forms for slip-form operation being erected

mortar, the mix being 1 part cement to 4½ parts sand.

Prestressing

After the tank wall and dome had been concreted, they were prestressed by winding with high tensile wire drawn to a diameter of 0.208 in. from the original diameter of 0.236 in.

The prestressing wire was applied by a wire-winding machine (Fig. 7) suspended along the side of the tank from a carriage which rode the outer edge of the dome. The carriage was anchored to the center of the dome by radial cables and the entire mechanism was self propelled.

The wall was prestressed to effect the pressure of tank contents and the dome was prestressed to withstand lateral dead and live loads of the dome itself. In prestressing the dome, wire was wrapped around the edge. The wire imparted an initial force of 326 kips per vertical ft at the base of the wall, decreasing gradually to a force of 30.4 kips per ft at the top. Prestressing force on the dome was 96 kips.

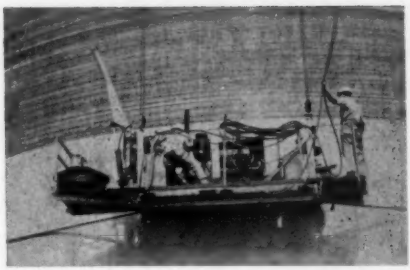


Fig. 7—Standpipe was prestressed by wrapping the wall and edge of the dome with high tensile wire applied by a wire-winding machine

About an inch of pneumatic mortar was applied over the exterior of the tank and dome edge to bond the wire and to protect it from the weather.

Credits

The master plan for the water expansion program was developed by Charles J. Kupper, consulting engineer, New Market, N. J., and the standpipe was designed and constructed by The Preload Co., Inc., New York.

Colloquim in Dresden on precast shells

The International Association for Shell Structures has received an invitation from Dr. H. Rühle, IASS executive council representative from the German Democratic Republic, to attend a colloquim on precast shells, which will take place at Dresden (DDR), probably the first part of November 1960. Dr. Rühle is organizing the meeting with the cooperation of the "Kammer de Technik."

In the last few years specialized technicians have been developing various solutions, each one with its advantages. Hence this meeting is deemed necessary to study, make known, and discuss the details of the prefabrication technology, as well as its problems dealing with the construction of shell structures.

The provisional program entitled "Prefabrication and the Possibilities of Its Application in Shell Structures," will present papers on three important phases of shell construction: (a) Fabrication of the elements: covering type of forms, materials used, number of reuses, time for making and demolding each element, treatment to accelerate the process of construction, sizes and weight or the appropriate elements; margins of error in the dimensions, minimum number of similar elements to reduce the costs, and related subjects. (b) Assembly: appropriate materials (concrete mortar, ceramics, metal, plastics); systems of transport and auxiliary elements for lifting and placing; use of post-tensioned wires; joining devices; centering devices and correction of positioning in situ. (c) Appropriate structural types: most appropriate shapes for the use of prefab-

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- **Absorption by Concrete of X-Rays and Gamma Rays**—A discussion of the mechanism of the absorption of x-rays and gamma rays by various shielding materials.
- **Properties of High-Density Concrete Made with Iron Aggregate**—Data on the physical properties of several types of mortar and concrete made with iron-bearing aggregate.
- **Heavy Steel Aggregate Concrete** — Gives consideration to various mix proportions for heavy concrete and a proportioning procedure for concrete of given strength and density.
- **Properties of Heavy Concrete Made with Barite Aggregates**—Presents data on test results on barite (barium sulfate) in conventional and prepacked concrete where high density is desired.
- **Magnetite Iron Ore Concrete for Nuclear Shielding**—Physical properties and costs of magnetite ore concrete are compared to those of other concretes.
- **Proportioning of Mixes for Steel Coarse Aggregate and Limonite and Magnetite Matrix Heavy Concretes**—A discussion of structural concrete utilizing limonite and magnetite ores as fine aggregate and graded steel scrap as coarse aggregate.

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rication; roofing materials and its waterproofing; economic studies.

Persons wishing to present a paper at this colloquim should contact Dr. Ing. H. Rühle, Dostojewskistrasse 10, Dresden-Loschwitz (DDR) enclosing a synopsis.

PCI convention program expands producers' sessions

Special sessions for the producer members of the Prestressed Concrete Institute will be expanded during the coming 6th annual convention to be held Sept. 27-30 at the Statler Hilton Hotel, New York.

The program committee has increased the amount of time and the number of sessions to be devoted to questions of specific interest to producers. Among topics of particular concern to this group are: combination pre- and post-tensioning versus draped strand, extrusion, automation and mechanization, and financial aspects of the prestressed business. Producers' topics will be discussed by a three-man panel with ample time for discussion.

Permanente Cement to build bulk cement depot

Permanente Cement Co., Oakland, Calif., will establish a bulk cement distribution plant in Eureka, Calif. Representing an investment in excess of $125,000, the plant will have a storage capacity of 3000 bbl of cement.

In addition to plant personnel, sales representative Lee Aldrich will be headquartered at the Eureka installation.

Permanente has cement producing plants at San Jose and Lucerne Valley, Calif.; Bellingham, Wash.; and a fourth plant currently under construction near Honolulu.

The Eureka facility will raise the number of Permanente distribution plants to eleven. The depots are strategically located throughout California, Oregon, Washington, Alaska, Hawaii, and Guam.

Story Behind the New Standard Sieve Specifications

By Albert E. Reed*

The publication by the American Society for Testing Materials of revised specifications for standard testing sieves to be known as E 11-58 is the crowning touch to the development of the U.S. sieve series that began 50 years ago.

Prior to 1909, testing sieves were pretty much a random affair. Those in use followed common "market grades" of available wire cloth such as 10, 20, 30, 40, 50 mesh, etc., with no pretension of logical relationships or even progression between the openings, some of which were closer together and others farther apart — just like a rule graduated with different size inches.

First fixed ratio sieve series

In 1910, the Tyler Standard Screen Scale Series using the opening of the 200 mesh sieve (which had been standardized for cement by the U. S. Bureau of Standards) as a base and progressing above and below this base in the fixed ratio of the square root of two was introduced.

This ratio, which had been suggested as far back as 1867 by Prof. P. R. von Rittinger in Germany, proved to be ideal for use in studying screening and crushing problems in ore mining and mineral aggregates for concrete construction.

In the square root of two series, the area or surface of each successive opening in the scale is just double that of the next finer or half that of the next coarser.

In other words, the width of the successive openings have a constant ratio of 1.414 while the area of the successive openings have a constant ratio of 2. Another advantage is that by using every other sieve, you have a ratio in the width of opening of 2 to 1. Later, when more sieves seemed desirable in the finer openings, intermediate sieves were added making the series according to the fourth root of two, this suggestion originating with Prof. Robert H. Richards of The Massa-

Sieving in Middle Ages, from "De Re Metallica" by Georgius Agricola, 1556

A—Sieve. B—Its handles. C—Tub. D—Bottom of sieve made of iron wires. E—Hoop. F—Rods. G—Hoops. H—Woman shaking the sieve. I—Boy supplying it with material which requires washing. K—Man with shovel removing from the tub the material which has passed through the sieve

*Vice-President, The W. S. Tyler Co. Abstracted from the Oct. 1959 issue of "The Testing World" published by Soiltest, Inc.

TABLE 1—NATIONAL STANDARDS IN THE FOURTH ROOT OF TWO GROUP

Country	Standards designation	Year
United States	Tyler	1910
United States	USBS and ASTM	1919
		1958 Revised
Great Britain	BS 410	1931
		1943 Revised
Canada	GP-1A	1937
		1953 Revised
		1958 Amended
Uruguay	U.N.I.T. 39	1944
Argentina	IRAM 1501	1950
Ireland	Irish Stand. 24	1950
Netherlands	N430	1952
India	IS 460	1953
Japan	JIS	1953
		1956 Revised
Australia, New Zealand, South Africa	Both British and ASTM series in use	

chusetts Institute of Technology and author of *Ore Dressing*.

The completed sieve series was formally presented to the scientific world by a paper delivered before ASTM in 1913 by G. A. Disbro.

In the Tyler series, the sieves were designated according to the meshes per inch of each sieve with the openings shown in inches and millimeters.

U. S. sieve series

In 1919, the U. S. Bureau of Standards announced a U. S. Standard sieve series based on the fourth root of two, using the 1 mm opening as a base, with micron designations for the sieves and sieve "numbers" as an optional designation. The openings in the U. S. series followed the Tyler series so closely that they could be used interchangeably, in fact, the labels of both the Tyler and U. S. series usually carry the equivalent designation in the other series.

Other national sieve standards

Following the success of Tyler and the U. S. series, other nations proceeded to adopt sieve standards based on the fourth root of two following closely the openings of the U. S. series. The nominal mesh designations were often different, as, for example, in the British series where the designations for many of the sieves were changed to make use of the British standard wire diameters. Since these variations in designation and wire diameter did not affect the openings, these standards were for all practical purposes interchangeable with the U. S. series.

Table 1 is a partial list of the national standards in the fourth root of two group in approximate sequence of their adoption.

A number of other countries have established sieve standards not based on the fourth root of two. Some European countries have been favoring the use of the 10th and 20th root of 10 as a basis for sieve series and these series contain sieves which coincide closely to some of the sieves in the fourth root of two Series. This fact makes it possible to compare sieve analyses made in countries using the fourth root of two with those made in countries using the 10th or 20th root of 10 series by using the points of coincidence in each series as a basis.

Problem of wire diameters

Back in 1909, when the Tyler series was being designed, it was necessary to make use of the best available combinations of weaving reeds and wire diameters to produce the exact openings required for each sieve. The result was that some specifications had wire diameters that were either heavier or lighter than desirable to produce a smooth uniform progression in opening-wire diameter relationship.

The introduction of the U. S. sieve series further complicated the situation by widening instead of narrowing the range of permissible wire diameters that could be used for each sieve. This difficulty has at long last been solved by ASTM E 11-58 (Table 1) which specifies a single wire diameter for each sieve opening and establishes tolerances of permissible deviation.

Fortunately, for Sieve users, the new ASTM E 11-58 sieve specifications which are now in production by the

leading manufacturers, do not make obsolete standard U. S. sieves now in use, since the openings in the new series remain the same.

The changeover in actual usage will be orderly. As the sieves in use and in dealers' stock wear out, they will be gradually replaced by sieves manufactured to the new ASTM specifications.

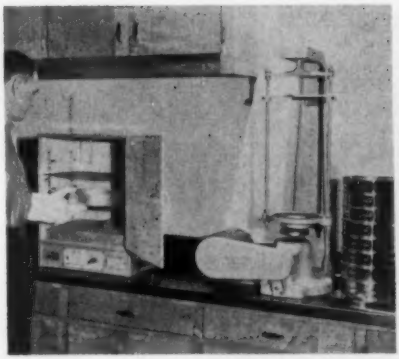
Toward a world standard

The International Standard Organization through Subcommittee TC/24-SC1 on Test Sieves has been intensifying its efforts to establish an international test sieve series.

Participating members of this committee are: Czechoslovakia, France, Germany, India, Netherlands, New Zealand, Great Britain, United States, USSR, Union of South Africa.

Observer members are: Austria, Belgium, Denmark, Hungary, Norway, Portugal.

It is earnestly hoped that the International Standards Organization will succeed in adopting a sieve series which will be acceptable to the majority of sieve users throughout the world so that technical data expressed in terms of sieve analyses may be freely exchanged and universally understood by the sieve users in all nations.



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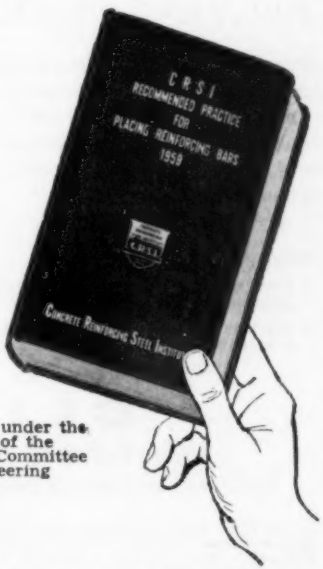

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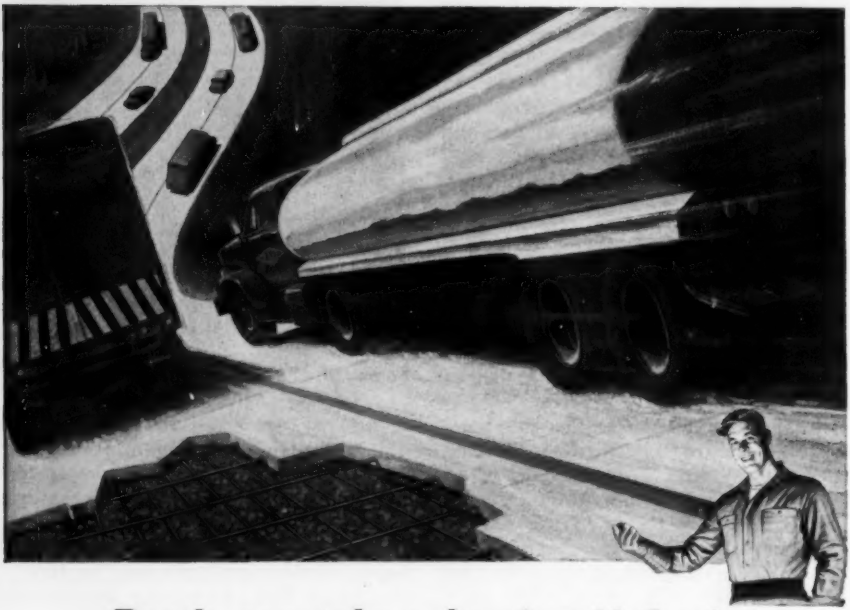

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Positions and Projects

Friel awarded honorary degree

Francis S. Friel, president, Albright and Friel, Inc., Philadelphia, was one of four men awarded honorary degrees at the 125th commencement at Lafayette College, Easton, Pa.

Mr. Friel was awarded a Doctor of Engineering Science degree, particularly because of his participation in international dam, power, and soil mechanics projects.

Mr. Friel was national president of the American Society of Civil Engineers in 1958-59. He is currently vice-president of the International Commission on Large Dams and from 1955 to 1958 was chairman of the U. S. Committee on Large Dams.

Geer accepts Thailand post

Elihu Geer, on leave from his position as professor and chairman, Department of Civil Engineering, University of Detroit, Detroit, has accepted a faculty position at the SEATO Graduate School of Engineering in Bangkok, Thailand. Professor Geer will be abroad for 2 years.

Universal Atlas announces research promotions

Universal Atlas Cement Division of U. S. Steel has announced promotion of two of its research personnel to newly created managerial positions.

Norman C. Ludwig, research associate at the division's laboratories, Buffington (Gary), Ind., has been appointed manager, Manufacturing Research, Raw Materials, and Manufacturing Division.

William V. Friedlaender, also research associate at the Buffington laboratories, has been advanced to the post of manager, Products Development and Quality Analyst, in the Quality and Technical Services Division at New York City.

Scientists from South Africa pursue research studies in U.S.

Under a contract with the International Cooperation Administration, the National Academy of Sciences–National Research Council of America is administering a program under which 150 young scientists at the immediate post-doctoral level are being brought to the United States to pursue basic research in American universities.

The program originated in 1953 when the Academy-Research Council was requested by the International Cooperation Administration to undertake the administration of such a program. The countries involved are those participating in the Organization for European Economic Cooperation (OEEC); these include Austria, Belgium, Denmark, Greece, Iceland, Italy, Luxemburg, the Netherlands, Norway, Portugal, Turkey, the United Kingdom, and West Germany. Since that date 216 scientists from these countries have participated in the program. In 1958 the program was extended to include South Africa and other free world countries to assist them to meet the worldwide demand for highly trained scientific personnel.

In administering the program the Academy-Research Council sought the cooperation of sister academies of sci-

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entists in each country to participate in the selection process.

In South Africa the Council for Scientific and Industrial Research undertook the sponsorship with the aid of a specially appointed selection committee. Of the nine applicants recommended by the committee, five were placed in the United States.

M. F. Kaplan, head of the Civil Engineering Division of the National Building Research Institute, was placed at the National Bureau of Standards at Washington, D.C., to work under Douglas E. Parsons, chief, Building Technical Division. A. G. Lutsch, senior research officer, National Physical Research Laboratory, was placed at the University of Providence, R. I., to work under Prof. R. B. Lindsay; S. J. Powell, lecturer, Department of Medicine, New Orleans, La., to work under Dr. J. C. S. Paterson.

A. M. Stephen, senior lecturer in organic chemistry at the University of Cape Town, was placed at the University of Minnesota to work under Prof. F. Smith; and J. H. N. Loubser, professor of physics, University of the Orange Free State, was placed at the Columbia Radiation Laboratory, New York City, to work under Professor Charles.

Public and Municipal Services Congress and Exhibition

The 17th Public Works and Municipal Services Congress and Exhibition will be held at Olympia, London, from Nov. 14-19, 1960. This project has been held in London biennially, except during the war years, since 1919, under the joint auspices of British departmental, public, and local authorities.

Internationally, the importance which the dominion, colonies, and other overseas countries attach to the congress and exhibition is constantly growing, and in 1958 representatives from 85 of them were in attendance.

The congress sessions are open to all visitors. Approximately 22 sessions are planned with experts in their respective fields participating in the program. The exhibition will feature approximately 350 exhibits.

Jidnal named to Punjab faculty

Roop Lall Jidnal has been named assistant professor of civil engineering at Punjab Engineering College, Chandigarh, India.

Professor Jidnal received his MS degree from the University of Illinois, Urbana.

Yee establishes consulting firm

Alfred A. Yee announces the establishment of Alfred A. Yee and Associates, structural engineering firm, with headquarters in Honolulu, Hawaii. Mr. Yee was formerly associated with Park and Yee, Ltd.

Technical seminar conducted at IIT

The Illinois Institute of Technology, Chicago, conducted an evening technical seminar on Design of Composite Steel and Concrete Beams for Bridges and Buildings on July 14.

The topics treated included theory, research, specifications, and design and construction practice.

Camellerie opens consulting office

J. F. Camellerie, formerly vice-president and chief engineer of Plummer Associates, New York, has opened his own consulting engineering office with headquarters in Huntington, N.Y.

Mr. Camellerie, a graduate of City College of New York, has been active for more than a decade in the design and construction of structures using the slip-form method. His paper, "Slip Form Details and Techniques" published in the April, 1959, JOURNAL, won him the ACI 1959 Construction Practice Award. He is currently serving on ACI Committee 714, Concrete Bins and Silos.

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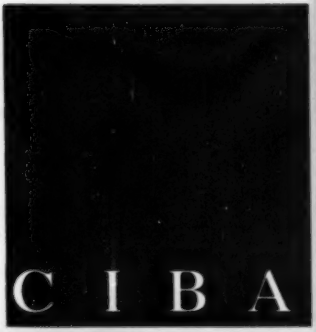

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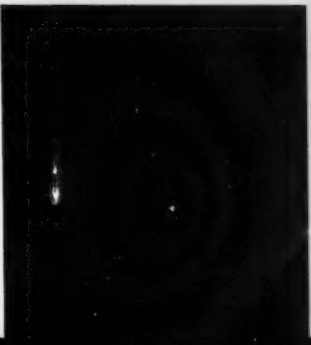

CIBA Products Corporation
Fair Lawn, New Jersey

Another New York skyscraper has been reinforced with USS American Welded Wire Fabric. This is Rockefeller Center's new forty-eight story Time & Life Building, an outstanding example of contemporary architectural design. The exterior steel columns are encased in stone-faced concrete which project from the walls and serve to accent the vertical sweep of the tower.

The frame supports short span, lightweight concrete slabs reinforced with USS American Welded Wire Fabric. Each slab is 8'0" long and 4" thick. When asked why the fabric-reinforced short-span design was selected for this structure, W. B. Scofield, partner in the structural engineering firm of Edwards & Hjorth, said "This system provides first-class, fireproof construction with a long record of satisfactory service in addition to its proven economy, speed of construction, and occupancy flexibility."

USS American Welded Wire Fabric was also used to reinforce the concrete fireproofing encasement of the columns, girders, and beams. Fabric is excellent for this application because the small, closely spaced members reinforce this thin concrete best. In addition, fabric is easily shaped to fit the contours and is sufficiently rigid to maintain the required shape.

Please write American Steel & Wire, Dept. 0331, 614 Superior Ave., N. W., Cleveland, Ohio or contact our nearest sales office for complete information on these or any other uses of USS American Welded Wire Fabric.

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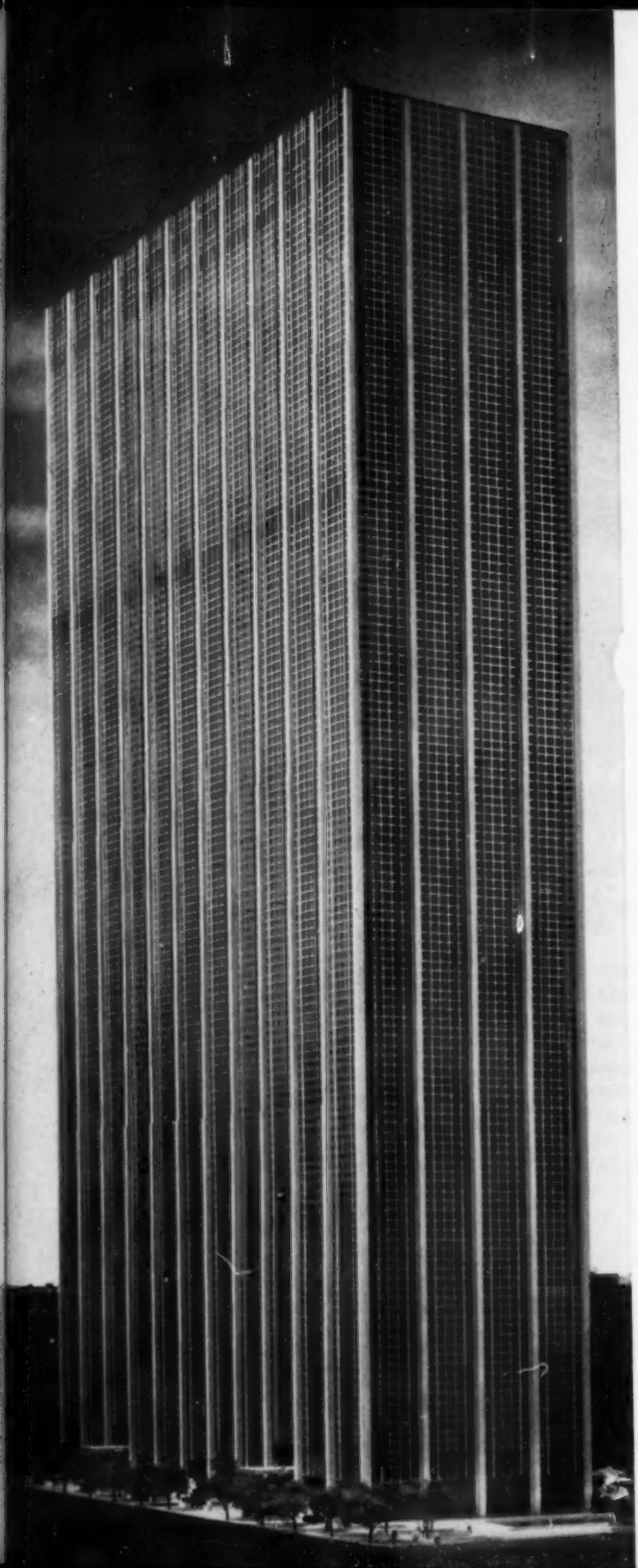
Short-span fabric-reinforced floor system in Rockefeller Center's Time & Life Building. American Welded Wire Fabric was furnished in long rolls and merely unrolled perpendicular to the beams and on top of the forms. It was draped from the top of the slab over the beams to the bottom of the slab at mid-span. Thus, the reinforcement is in position to best resist both positive and negative moments. The economy of steel placement is apparent. In total, over six million square feet of short span slabs reinforced with Welded Wire Fabric have been used in New York's Rockefeller Center.

Owners: Rockefeller Center, Inc. and Time Inc.
Architects: Harrison & Abramovitz & Harris
Structural Engineers: Edwards & Hjorth
General Contractor: George A. Fuller Company, John Lowry, Inc.
Fabric Distributor: Fireproof Products, Inc.



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ASTM holds 63rd annual meeting

Few areas of materials research and standards were left untouched during the week of the American Society for Testing Materials 63rd annual meeting at Chalfonte-Haddon Hall, Atlantic City, N.J., June 26-July 1.

Approximately 3000 members, committee members, and guests attended the 37 technical sessions covering topics ranging from the most basic of research in materials sciences to the most practical aspects of engineering design and construction. ASTM technical committees and their subcommittees held over 900 meetings during the week. The society's biennial exhibit of testing apparatus and laboratory supplies was held in connection with the annual meeting.

Officers for 1960-61

The election of new ASTM officers was announced during the luncheon on June 28.

A. Allan Bates, vice-president of research and development, Portland Cement Association, Chicago, was elected president to serve for a 1-year term beginning at the close of the meeting. Russell Wade Seniff, manager of research, Baltimore and Ohio Railroad, Baltimore, Md., was elected vice-president. Miles N. Clair, president, The Thompson and Lichtner Co., Brookline, Mass., will continue as senior vice-president of the organization.

New members of the board of directors elected for 3-year terms were: Arthur R. Belyea, Consolidated Edison Co. of New York, Inc., New York; Leslie V. Cooper, Firestone Tire and Rubber Co., Akron, Ohio; Jay C. Harris, Monsanto Chemical Co., Dayton; Jerome J. Kanter, Crane Co., Chicago; Robert F. Legget, National Research Council of Canada, Ottawa, Ont.; and William R. Willets, Titanium Pigment Corp., New York.

Dr. Bates, a scientist and chemical engineer with broad experience in industrial research, has been an ASTM member since 1946 and has served on the ASTM board of directors since 1955. He has participated in ASTM committee work for many years as representative of PCA on Committee C-1 on Cement and in connection with Committee C-9 on Concrete. As chairman of the C-1 subcommittee on the Cement Reference Laboratory, Dr. Bates has been especially active in developing the operations of this laboratory which involve the coordinated interests of many private and governmental groups and agencies interested in testing of cements and concretes.

Long active in ACI affairs, Dr. Bates is currently a member of the Institute Board of Direction. He also served on the board from 1952-54. Assuming chairmanship of the ACI Building Committee in October, 1959, he steered the committee through a successful campaign, culminating in the ACI headquarters building.

Dr. Bates holds degrees from Ohio Wesleyan University and Case Institute of Technology and a DSc from

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RECOMMENDED PRACTICE FOR HOT WEATHER CONCRETING (ACI 605-59) spells out ways to minimize detrimental effects of hot weather on concrete. Means are described for reducing concrete temperatures by proper attention to ingredients, methods of production and delivery, and care in placement, protection, and curing. The use of admixtures to reduce mixing water requirements and to retard setting is also covered. 10 pp. Price $1.00, 50¢ to ACI members.

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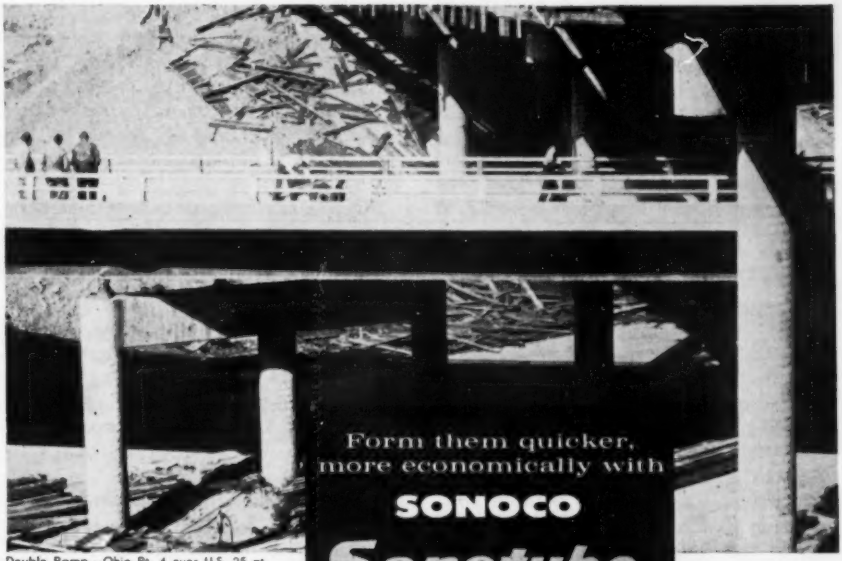

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the University of Nancy in France. He is active in a number of other technical societies.

Awards

Raymond E. Davis, professor emeritus of civil engineering at the University of California, Berkeley, was presented the Frank E. Richart Award. This award, established by Mrs. Richart in memory of her late husband, is presented not oftener than every 3 years to a committee member of the society who has made a notable contribution to the work of the organization in the field of concrete and concrete aggregates.

Professor Davis, past president and honorary member of the American Concrete Institute, has been associated with the University of California since 1920 as professor of civil engineering and director of the Engineering Materials Laboratory. He retired from active teaching in 1953.

Professor Davis is noted for his work in research in the field of cements, aggregates and concretes, and in the design and construction of concrete structures, principally mass concrete such as dams. During his long career he has served as consultant to various government agencies and private corporations. He is the author or coauthor of nearly 100 papers and discussions. ACI has awarded him the Henry C. Turner Gold Medal, the Construction Practice Award, and twice the Wason Medal. He is currently vice-chairman of ACI Committee 115, Research, and a member of ACI Committee 207, Properties of Mass Concrete.

Stanton Walker and Delmar L. Bloem, director and associate director of engineering, respectively, National Sand and Gravel Association and the National Ready Mixed Concrete Association, received the Sanford E. Thompson Award for their paper on "Variations in Portland Cement." This annual award is presented to the author or authors of a paper, published by ASTM, of outstanding merit in the field of concrete and concrete aggregates.

Mr. Walker, past president and honorary member of the American Concrete Institute, has been director of engineering of the National Sand and Gravel Association since 1926, and since 1930 director of engineering of the National Ready Mixed Concrete Association. He is also director of the Joint Research Foundation of NSGA and NRMCA at the University of Maryland.

In addition to his activities with ASTM and ACI, Mr. Walker is past chairman and member of the executive committee of the Highway Research Board, and a member of the AIME. He is the recipient of the Roy W. Crum Award of the Highway Research Board, the ASTM Award of Merit and Frank E. Richart Award, and is author of numerous papers dealing with research in concrete and mineral aggregates.

Mr. Walker is currently a member of ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes, and Committee 621, Aggregates.

Delmar L. Bloem joined the National Sand and Gravel Association and National Ready Mixed Concrete Association staff in 1945 and was appointed to his present position as associate director of engineering in 1954.

Since joining ASTM in 1954, Mr. Bloem has been extremely active and has served on a number of ASTM committees as well as on the Washington, D.C., District Council. He is the author of many technical papers and research reports.

In addition to his membership in ASTM, Mr. Bloem is a member of ACI,

the Highway Research Board, AREA, and ASCE.

Active in ACI Committee work for a number of years, Mr. Bloem is currently chairman of ACI Committee 116, Nomenclature, and a member of four other ACI technical committees.

Three ACI members were among 19 leaders in the field of engineering materials honored with the society's Award of Merit: Harold F. Clemmer, engineer of materials and standards, District of Columbia Highway Department, Washington, D.C., in recognition of long and effective services to ASTM technical committees, district and administrative activities, and especially for work in Committee D-4 on Road and Paving Materials, and other technical committees; Harry C. Plummer, director of engineering and technology, Structural Clay Products Institute, Washington, D.C., in recognition of long-time constructive participation in the work of ASTM building materials committees, especially C-7 on Lime, C-12 on Mortars for Unit Masonry, and C-15 on Manufactured Masonry Unit, and for promoting the use of ASTM standards by architects and engineers; Charles H. Scholer, consultant, formerly head, Department of Applied Mechanics, Kansas State College, Manhattan, in recognition of many years of sustained productive service in ASTM technical committee work, especially as a member of Committee C-1 on Cement, Committee C-9 on Concrete and Concrete Aggregates, and Committee D-4 on Road and Paving Materials.

Calaveras transfers Lindgren to Oregon

The transfer of Charles Lindgren from Fresno, Calif., to Eugene, Ore., as sales representative for Calaveras Cement Co., a division of The Flintkote Co., was recently announced. His transfer follows the company's opening of a new bulk cement transfer plant at Springfield, Ore., to serve the 13 southern counties of that state.

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Errata

The following corrections should be made in the "Current Reviews" section of the March, 1960, ACI JOURNAL.

p. 896—add the following to the first sentence of the review "Plastic buckling of the circular-cylindrical shell" so it reads—". . . the radial and axial directions; isotropic: identical moduli in the radial and axial directions)."

p. 897—change the word in the first sentence of the review "Analytical stress determination for tunnel cross sections" from ". . . conformed . . ." to ". . . conformal . . ."

* * *

The following correction should be made in the review "Damaging reactions in concrete," on p. 1084-1085 in the "Current Reviews" section of the April, 1960, ACI JOURNAL.

p. 1084-1085 — change the fourth sentence of the third paragraph to read, in part, ". . . for reactive aggregate, nor do admixtures, proved to be effective, work in this case."

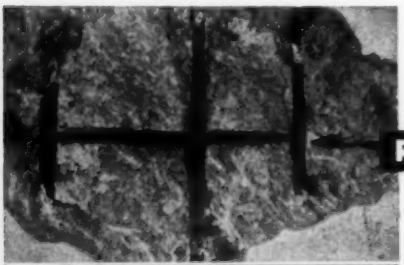
Stubbs joins faculty at University of California

F. W. Stubbs, Jr., formerly professor of civil engineering, Purdue University, Lafayette, Ind., has joined the civil engineering staff at the University of California, Berkeley.

Quinn appointed to Master Builders' Seattle branch

Albert S. Quinn, Jr., has been named sales representative for the Master Builders Co.'s Seattle branch office.

Prior to joining the company, Mr. Quinn was associated for the past 4 years as a sales engineer with the Stebbins Engineering Corp., Seattle.



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NATIONWIDE SERVICE

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

Individual Members (U.S. and Possessions, Canada, Mexico, Central America, and West Indies)	**$20.00**
Individual Members (All other foreign countries)	**16.00**
Corporation Members	**65.00**
Junior Members—nonvoting (under 28)	**10.00**
Contributing Members	**135.00**
Student Members—nonvoting (under 28)	**5.00**

Please enclose remittance with application (cut here)

Board of Direction, American Concrete Institute **Date**_____
P. O. Box 4754, Redford Station
Detroit 19, Michigan

The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter) _____

_____

For Corporation Membership, ACI representative will be _____

Date of Birth (Juniors and Students only) _____
Year Month Day

College or University attending (Students only) _____

_____ _____
(Date of graduation) (Proposed by)

Signature_____

For our records, please complete both sides of application.

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

Classification is not based on what interests you, but on what you do—what office or job you fill. When completing the form below, please check the one item in each section that is most applicable to that particular section.

ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

MEMBER CLASSIFICATION

JOB TITLE _____

OCCUPATION (Check the one most applicable)

☐ Arch ☐ Engr ☐ Construction Supervision ☐ Plant Management or Supervision ☐ Teaching ☐ Student ☐ Other (please state) .....

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☐ Architect ☐ Contractor ☐ Consulting Engr ☐ Engr Firm ☐ Manufacturer or Producer (specify product) .....

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☐ Other (please state) .....

Do you ☐ Specify ☐ Authorize ☐ Recommend, purchase of materials or equipment?

Honor Roll

Jan. 1—June 30, 1960

Let's keep the membership trend spiraling. YOU can further the growth and prestige of the Institute by proposing new members. Tell your colleagues about ACI activities! Add your name to the Honor Roll next month.

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Alfonso Marin E. 33
Samuel Hobbs 32
Robert P. Witt 17
Harry Ellsberg 15
Joaquin Spinel L. 14
Walter H. Price 12
C. P. Siess 11
Faraj Tajirian 10
Roger H. Corbetta 9
Alfonso Golderos 9
James A. McCarthy 9
Gene M. Nordby 9
J. Raymond Watson 9
Milton H. Zara 9
J. F. Toppler 8
Yoshikatsu Tsuboi 8
James Chinn 7
W. S. Cottingham 7

Ward W. Engle 7
Napoleon Ferrer G. 7
Russell Porter 7
Pastor B. Tenchavez 7
J. Karni 6½
S. O. Asplund 6
Ernst Basler 6
Jacob J. Creskoff 6
Evan Savours David 6
H. C. Delzell 6
Joseph J. Fox 6
Arturo Guevara 6
Antonio A. Henson B. 6
T. C. Kavanagh 6
John C. McCoe 6
Jose Luis Montemayor 6
W. E. Moulton 6
John D. Smith 6
George B. Southworth 6
R. H. Wildt 6
C. A. Willson 6
Pedro M. Bassim 6

John E. Heer, Jr. 5
Adrian Pauw 5
H. C. Pfannkuche 5
Felix Colinas Villoslada . 4
John G. Dempsey 4
Phil M. Ferguson 4
G. Grenier 4
Martin J. Gutzwiller 4
William W. Karl 4
Joe W. Kelly 4
Narbey Khachaturian ... 4
Henry A. Lepper, Jr. 4
Jack Longworth 4
Willard A. Oberdick 4
Gerald F. Paulson 4
Ingvar Schousboe 4
John Adjeleian 3
Michael Alexander 3
James E. Amrhein 3
Amos Atlas 3
Salvatore Azzaro 3
J. E. Backstrom 3
E. E. Barreiro M. 3
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Luther E. Bell 3
O. R. Bell 3
M. R. Berretti 3
Dan E. Branson 3
Martin R. Brown 3
R. C. Brown 3
Allen H. Brownfield 3
Vincent R. Cartelli 3
Alan Carter 3
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Walter F. Conlin, Jr. 3

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Fernando De Angulo 3
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Hans Gesund 3
H. J. Gilkey 3
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Seymour W. Greenberg . 3
Fabian Guerra 3
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Kenneth Hansen 3
Warner Harwood 3
J. T. Helsley 3
Norman E. Henning 3
Aleck E. Hiscox 3
Roy Holte 3
Robert H. Hopwood 3
H. Y. Hsu 3
Kenneth M. Huber 3
G. M. Idorn 3
Mario Jimenez-Cadena .. 3
E. H. Johnson 3
H. Alan Johnson 3
Oliver G. Julian 3

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Karl Kaspin 3
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Ignacio Martin 3
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J. Antonio Thomen 3
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Alex Tobias 3
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D. A. Van Horn 3
Fernando Vega 3
Ellis S. Vieser 3
Sam Walden 3
E. H. Walker 3
Carl Weber 3
Cedric Willson 3
J. A. Wineland 3
Eugene C. Wong 3
T. W. Wood 3
Mark R. Woodward 3
Jayantilal S. Alagia 2
Arthur H. Andersen 2
Vahe Aprahamian 2
Carlos Asturias P. 2
Charles O. Baird, Jr. 2
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H. J. Brettle 2
D. Campbell-Allen 2
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Guillermo Castellanos G. 2
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William G. Corley 2
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Melton A. Croson 2
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Enrique Garcia-Reyes .. 2
M. O. Glew 2
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Gregorio Hernandez 2
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Raja A. Iliya 2
Fred W. Jacobs 2
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Oscar Latorre M. 2
G. H. Matchette, Jr. 2
Howard R. May 2
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Alvaro Villagas Moreno.. 2
R. R. Neal 2
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Roberto E. Prata Lou ... 2
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Jose H. Rizo 2
Andrew W. Rose 2
Walter D. Rudeen 2
Russell Schofield 2
E. C. Sword 2
Alatz Quintana Uranga . 2
J. M. Warne 2
Leon A. Yacoubian 2
Roger M. Zimmerman .. 2
Antonio Zuniga Ayala .. 2

New Members

The Board of Direction approved applications in the following categories: 65 Individual, 3 Corporation, 14 Junior, and 10 Student, making a total of 92 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,300.

INDIVIDUAL

ADAMS, A. BYRON, Cambridge, Mass. (Tech. Dir., W. R. Grace & Co.)
ADKINS, WILLIAM H., Tulsa, Okla. (Struct. Engr.)
AEDO C., Francisco, Santiago de Chile (Dir., Inst. de Edificacion Experimental)
ANDRES, OSCAR A., Bahia Blanca, Argentina (C. E.)
BARBER, J. BYRON, Spokane, Wash. (Struct. Engr., H. E. Bovay, Jr., Cons. Engrs.)
BAUMGART, NORMAN D., Helena, Mont. (Dist. Engr., PCA)
BELLOWS, G. F., Houston, Tex. (Vice-Pres., W. S. Bellows Constr. Corp.)
BESS, TOM D., Tulsa, Okla. (Struct. Engr.)
BLAYLOCK, ALBERT J., San Diego, Calif. (Owner, C. E. & Assocs.)
BURK, PETER W., JR., Honolulu, Hawaii (Tech. Ser. Engr., Hawaiian Cement Corp.)
CHATTERJEE, B. K., Calcutta, India (Part., Chatterjee & Polk)
CHEN, MALCOLM (Mien Chee), Lindfield, NSW, Australia (Engr., NSW Dept. of Main Rds.)
COWE, DAVID, Hamilton, Ont., Canada (Insp., Roadway Div., City of Hamilton)
CRACRAFT, DONALD K., Long Beach, Calif. (Sr. Engr., Bechtel Corp.)
CUNNINGHAM, J. R., Freeport, Tex. (Civil Des. Engr., Dow Chem. Co.)
DELANO, DALE C., Okla. City, Okla. (Chf. Engr., Thomas Conc. Pipe Co.)
DINOS, COSTANTI, Hato Rey, P. R. (Struct. Engr., Dinos & Vafi, Cons. Engrs.)
ERIKSSON, CARL EINAR, Columbus, Ohio (Chf. Brdg. Engr.)
ERIKSSON, OWE, Beaumont, Tex. (Assoc. Prof., Lamar State Coll. of Tech.)
EVANS, E. R., Burlington, Ont., Canada (Sr. Struct. Engr., L. H. Schwindt & Co., Ltd.)
FAIRN, LAIRD, Halifax, N. S., Canada (Arch., Leslie R. Fairn & Assocs.)
FERRER, JESUS R., JR., San Juan, Rizal, Philippines (Proj. Engr.)
FISCHER, FRED, JR., New York, N. Y. (Part., Severud-Elstad-Krueger Assocs.)
FLAY, GEORGE F., JR., Mamaroneck, N.Y. (Chf. Engr., Stock Constr. Corp.)
GALLANTI A., MIGUEL AUGUSTO, San Cristobal, Venezuela (Struct. Engr.)
GARY, W. T., Augusta, Ga. (Plant Mgr., Engr., Augusta Conc. Prods. Co., Inc.)
GLADDING, RAYMOND D., Fontana, Calif. (Part., E. F. Gladding Co.)
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GRIFFIN, BURRELL D., Okla. City, Okla. (Lab. Mgr., Std. Testing & Engrg. Co.)
GRINBERGS, ALFRED, Chicago, Ill. (Hd., Conc. Dept., Meissner Engrs., Inc.)
HANSEN, E. I., Durban, Natal, South Africa (Engrg. Dir., Sayle & Rossaak, Ltd.)
HERB, E. G., San Francisco, Calif. (Exec. Vice-Pres., Intl. Engrg. Co., Inc.)

HILLMAN, SAMUEL, Camden, N. J. (Prod. Mgr., Vacuum Conc. Corp. of Am.)
HODGSON, R. A., Lagos, Nigeria (Struct. Engr., Ove Arup & Parts.)
HORN, LEONARD H., Long Beach, Calif. (Sr. Proj. Engr., Rockwin Engrs.)
HUBER, LNDY L., Austin, Tex. (Mgr., Conc. Prod. & Sales)
JAYASOORIYA, L. S. D. S., London, England (C. E., C. S. Allott & Son, Cons. Engrs.)
KAFAROWSKI, GRANT, Richmond Hill, Ont., Canada (Chf. Engr., Murray Assocs., Ltd.)
KAPLAN, M. F., Pretoria, South Africa (Rsch Engr., South Africa Council for Scien. & Indust. Rsch.)
KENNEDY, D. J. LAURIE, Urbana, Ill. (Rsch. Asst., Univ. of Ill.)
KNIGHT, STEPHEN C., JR., Burlington, Vt. (Assoc. Prof., Univ. of Vt.)
KURTZ, DONALD R., Flint, Mich. (Owner, Kurtz Gravel Co.)
LITTLE, L. W., Carson City, Nev. (Chf. Matls. & Test. Engr., Dept. of Hwys.)
MacDONALD, J. W., Halifax, N. S., Canada (Chf. Engr., N. S. Light & Power Co., Ltd.)
MANDELBERG, MICHAEL, Givataim, Israel (Deputy Hd., Des. Dept., Water Ping. for Israel, Ltd.)
McNEAL, RAY M., Belleville, N. J. (Gp. Ldr., Sonneborn Chem. & Refining Corp.)
MELITAS, A. C., Verdun, Que., Canada (Des. Engr., City of Montreal, Dept. of Pub. Wks.)
MORGAN, AUSTIN H., JR., Winchester, Va. (Tech. Ser. Engr.)
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PAPAMATHEAKIS, NICHOLAS, N. Balwyn, Vic., Australia (Des. Engr.)
PENNELL, JOHN F., San Andreas, Calif. (Rsch. Chem., Calaveras Cemt. Co.)
PENSHORN, HARVEY, San Antonio, Tex. (Des. Engr.)
PERKINS, LEONARD R., JR., Richmond, Va. (Qlty. Control Engr., Sn. Matls. Co., Inc.)
PLACAS, GEORGE, Montreal, Que., Canada (Struct. Engr., Brouillet & Carmel)
SMITH, N. DENHAM, Ridgeland, Miss. (Chem., Hunt Process Corp.-Sn.)
SPADA, RICHARD T., Chicago, Ill. (Salesman, W. R. Grace & Co.)
STORMER, JOHN C., Earlville, N. Y. (Gen. Mgr., Mid-State Conc. Plank, Inc.)
SUMNER, FRED L., SR., Pocatello, Idaho (Plant Mgr., Pocatello Ready Mix, Inc.)
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TOWNSEND, EDWARD E., Calhoun, Tenn. (Hd., C. E. Sect., Bowaters Engrg. & Dev., Inc.)
VEITAS, ROMUALDAS M., Pittsburgh, Pa. (Sr. Desr.)
WALSH, ROBERT J., Toronto, Ont., Canada (Constr. Engrg. Insp., James F. MacLaren Assocs.)

CORPORATION

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DILLY CONSTR. CO., Rapid City, S. D. (Boyce R. Coleman, Constr. Engr.)
NORTHERN LIGHTWEIGHT AGGREGATE, INC., Cohoes, N. Y. (Cyril F. Howe, Sales Mgr.)

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OLSON, RONALD H., Riverside, Ill. (Struct. Engr.)
POSADA P., CARLOS, Medellin, Colombia (Constr. Supv., Gerente de Estudios y Construcciones, Ltd.)
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WHEELER, ROBERT A., Atlanta, Ga. (Engr., J. A. Jones Constr. Co.

STUDENT

CAYADO, JULIO SANTOS, Trujillo City, Dominican Republic (Santo Domingo Univ.)
FATULA, JOHN A., State College, Pa. (Pa. State)
GIL Y ARIAS, STELLA, Bogota, Colombia (Bogota National Univ.)
GROSS, JEROME HIRSH, Van Nuys, Calif. (Univ. of Ariz.)
GUPTA, SATYA PARKASH, Lafayette, Ind. (Purdue Univ.)
LEVANTROSSER, FREDERICK C., Detroit, Mich. (Wayne State)
MAIZ G., JOSE ANTONIO, Monterrey, N. L., Mexico (Inst. Tecnologico y de Estudios Superiores de Monterrey)
MENDOZA-DURAN, JULIO, Bogota, Colombia (Universidad Nacional)
SIACA, RAFAEL DAVILA, Santurce, P. R. (Mayaguez Coll. of Agric. & Mech. Arts)
VITOLS R., JURIS, Caracas, Venezuela (Universidad Central Caracas)

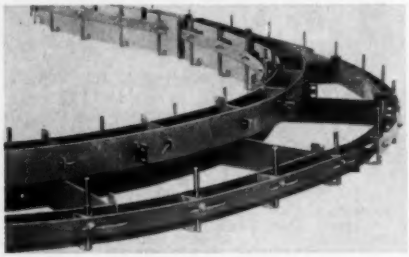
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Flexible steel form

Radi-Lok, a flexible steel form for placing uniform curves in concrete, produces smooth attractive facings for both inside and outside curves on low walls, sidewalks, curbs, and gutters; shortens setup time, minimizes stripping effort, and practically eliminates surface finishing according to the manufacturer.

Lightweight, but strong, manufacturer states that Radi-Lok will withstand constant



hard use and casting stresses. Comes in convenient 10-ft lengths with heights ranging from 4 to 14 in. They may be joined in series as shown in photo to make up a variety of contours.—Binghamton Metal Forms, Box 848, Church Street Station, New York, N.Y.

New cement for block

A cement specially formulated for the concrete block and products industry has been introduced by Permanente Cement Co.

The product is light in color, and tests conducted according to ASTM codes as well as tests under manufacturing conditions have determined that it achieves strength rapidly and has excellent workability.

Development of the product cement coincides with a major Permanente Cement promotion aimed at increasing the use of concrete block, particularly of the many new decorative and architectural designs.—Permanente Cement Co., Kaiser Bldg., Oakland 12, Calif.

Machine casts concrete pipe directly in trench

A machine that casts concrete pipe directly in the trench was demonstrated at the construction site of 8000 ft of storm drainage pipe for the city of Sunnyvale, Calif.

The No-Joint Machine, essentially a round-bottomed "boat," is lowered into the trench which is dug to conform to the outside diameter of pipe size. Ready-mixed concrete is placed into a hopper as the machine moves forward in the trench extruding the formed pipe in its wake. A gasoline engine mounted at the front of the machine provides the power for winching forward and for vibration and compaction of the concrete as it is pressed through to form the pipe.

During construction the upper half of the pipe is supported by sectional aluminum forms which are placed as the machine moves along and the trench itself forms the lower half. The aluminum sections are removed and made ready for re-use after initial set of the concrete.

Pipe sizes range from 24 in. to 6 ft in diameter. The weight of the No-Joint machines vary from 2 to 5 tons, depending on the diameter required. According to the manufacturer, the pipe can be cast in place at speeds up to 120 ft per hr, depending on size.—Monolithic Pipe Lines, Inc., P.O. Box 356, Station A, Palo Alto, Calif.

Surface coating

Decor-Cem, a decorative surface coating for interior and exterior walls, is pressure applied to cast concrete, concrete block, brick, or tile, over a wet base coat to produce a bonded single surface.

It is said that Decor-Cem withstands severe thermal shock and abrasive tests and is suited for every climate condition. It is also claimed to be waterproof, crack resistant, and fadeproof. Available in white, deep tones, or pastels; flat or glossy finish; smooth or textured surface.—Decor-Cem Inc., 1818 Broadway, Rockford, Ill.

Corrosion resistant topping

A. C. Horn Companies have developed a two-component floor topping and patching compound called Thiopoxy 60. It contains flexible Thiokol liquid polymers and epoxy resins designed to withstand severe corrosive and abrasive conditions which tend to destroy or disintegrate ordinary concrete. It is a

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Prepared by the Committee on Engineering Practice

CONCRETE REINFORCING STEEL INSTITUTE

38 S. Dearborn St. (Div. J), Chicago 3, Illinois

one-application compound applied by trowel, to a ⅛ in. thickness or more and finished by mohair paint roller, to interior or exterior, new or old concrete.

Developer claims it cures overnight, at normal room temperatures, to a dense, slip-resistant, nonshrinking, topping with impact and abrasion resistance.

It is recommended for repairing cracks and grouting the joints of acidproof brick and quarry tile floor, and as a covering on adjoining vertical surfaces, curbing, acid trenches, and solvent or alkali sewers.—Sun Chemical Corp., 750 Third Ave., New York, N.Y.

Tramp iron removal from slag

A recent development for removing tramp iron and iron bearing slag from the raw crushed slag feed uses a Memco SE series magnet, mounted 15 in. above the belt, obviating the need for fine iron separation later in the system.

A typical installation at a large steel mill incorporates the Memco SE series rectangular double gap suspended magnet arranged for cross belt operation. The unit is complete with reversible motor so that iron and iron-bearing slag can be discharged to either side of the main conveyor belt according to the user's preference.

The heart of the SE series magnet is the space winding feature and oil cooling of the coil, which assures higher operating wattage according to the manufacturer. The SE series magnets are available in all standard belt widths; in seven strengths for self cleaning or non-self-cleaning installations.—Magnetic Engineering and Manufacturing Co., 840 Van Houten Ave., Clifton, N.J.

Enclosed hopper trailer

Development of an enclosed Mono-Shell hopper trailer has been announced by Gar Wood Industries. The trailer has been designed for the hauling of bulk materials such as cement, sugar, and other products which must be protected from the elements.

The Mono-Shell design makes possible, according to the manufacturer, the largest

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Swimming pool coating

Plastic Armor SWF, a protective and decorative coating with improved underwater resistance for concrete pools, has been developed by Permagile Corp. of America.

The coating is said to have remarkable adhesion, durability, and chemical resistance properties. Drying does not depend on oxidation and/or evaporation of solvents and the resultant thermoset protective coating will not redissolve in solvents and other liquids according to Permagile spokesman.

The manufacturer states that when applied with brush or roller, two individual coats should be sufficient; when applied by spray, three individual coats are recommended.— Permagile Corp. of America, 37-23 Thirty-third St., Long Island City, N.Y.

Rubber roofing for thin shell concrete

Twinsburg-Miller have developed a system of roofing over thin shell concrete which they claim provides functional waterproofing as well as a decorative surface.

The system employs the use of butyl rubber in combination with woven glass cloth which results in a tough rubber skin with long-term life expectancy according to T-M spokesman.

It is further stated that it will not crystallize on aging, and does not check, alligator, or delaminate. The rubber skin provides freedom of movement without fracture should the concrete surface develop structural cracks or flexural movement.

The use of marble chips in complementary colors may be used to provide texture to the finished surface. Standard colors available are red, green, blue, gray, and white.—Twinsburg-Miller Corp., P.O. Box 207, Twinsburg, Ohio

Indicator and single unit totalizer

Remote indication of both batch and cumulative weights is given in a compact unit introduced by Richardson Scale Co. The unit combines indicator and single unit totalizer.

Totalizer records cumulative weight in pounds either of a single ingredient or, when used in a proportioning system, of a combination of ingredients. The indicator pointer, remote from the dial scale, follows the dial scale pointer movement. The standard counter contains six digits and has a reset feature. Available with dust-tight enclosure or may be panel-mounted. — Richardson Scale Co., Clifton, N.J.

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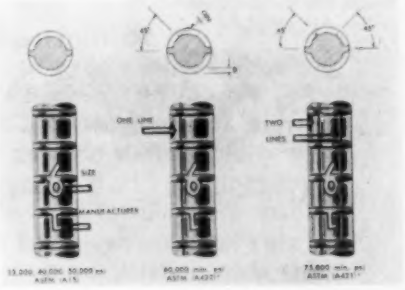
Marked reinforcing bars

Responding to the requests of many engineers, high-strength multirib reinforcing bars, carrying rolled-in markings for immediate identification, are now being produced by Laclede Steel Co.

The bars are identified for grade by one and two longitudinal ribs, respectively, rolled



into the bars at the time of manufacture. The identification system also includes a number designating the bar size and the letter "L" for company of origin.

The new Laclede bars conform to two new ASTM standards: A-432 for 60,000 psi minimum yield point steel; and A-431 for 75,000 psi minimum yield point. — Laclede Steel Co., St. Louis, Mo.

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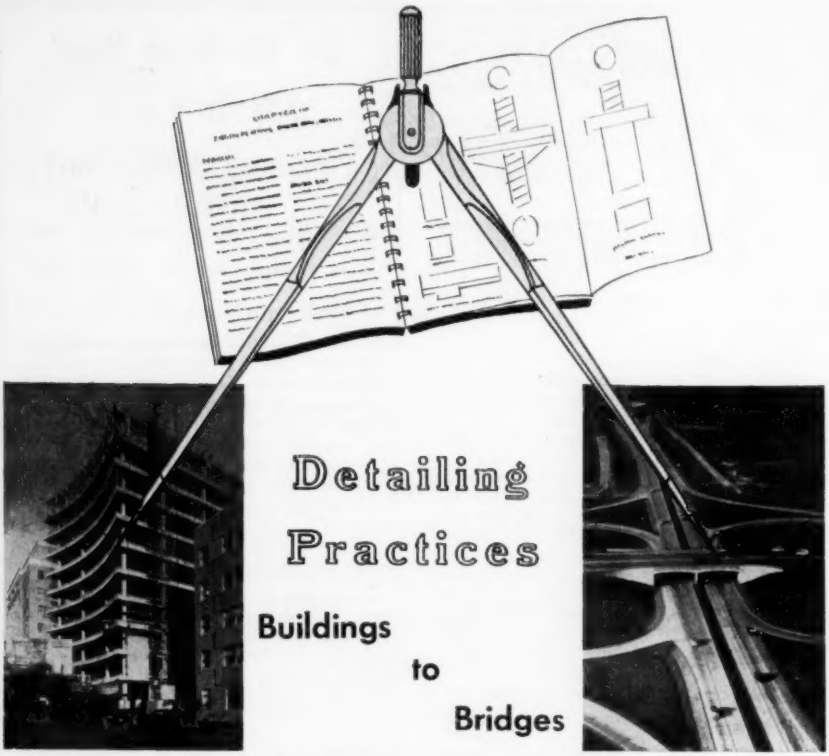

58 Illustrative Drawings

The ***Manual of Standard Practice for Detailing Reinforced Concrete Structures*** **(*ACI 315-57*)** incorporates under one cover the previous separate editions on detailing of buildings and highway structures. It is a correlation of the latest improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. Sections on detailing and fabricating shop practice are translated into practical examples in typical drawings. Spiral bound to lie flat, it is the only publication of its kind in English and is meeting wide acclaim among designers, draftsmen, and engineering schools.

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

WATER-REPELLENT MASONRY FILL—Two booklets describing the advantages and applications of Zonolite water-repellent masonry fill insulation for brick cavity and concrete block walls. The booklets, one on cavity walls and one on block walls, contain information and charts on the product's thermal efficiency, estimated fuel and air conditioning savings, coverage tables, water-repellency advantages, and ease of installation.—Zonolite Co., Inquiry Department, 135 S. LaSalle St., Chicago, Ill.

MASONRY-BONDING AGENT CATALOG — Six-page catalog covering Hornbond, a liquid resinous emulsion bonding agent for concrete, plaster, stucco, gypsum, cinder block, and tile, with a complete description of the product, its color, properties, function, method of application, coverage and tests for vibration, acidity and toxicity together with packaging information and architect's specifications.—A. C. Horn Companies, 2133 85th St., North Bergen, N.J.

BUCKET ELEVATORS—Catalog covers both belt and chain elevators in single and double casing design. Three standard series are included in a range of capacities from 280 to 8835 cu ft per hr. Recommendations are given on the types of elevators best suited for handling of 65 different materials.—Andrews Machine Co., 359 East Main St., Decatur, Ill.

BUILDING WITH PRESTRESSED CONCRETE — A 20-min sound-color film illustrates the ease of manufacture and the many advantages of prestressed concrete in the building industry. The film is available for showings to contractors, architects, engineers, government agencies, and schools.—Colorado Fuel and Iron Corp., P.O. Box 1920, Denver, Colorado

HORNCURE CONCRETE CURING COMPOUNDS—Brochure gives complete descriptions of Horncure 100 percent resin base, white pigmented, and wax resin base types of concrete curing compounds. Cites advantages, uses, and specifications.—A. A. Horn Companies, Division of Sun Chemical Corp., 2133 85th St., North Bergen, N.J.

NALCO SODIUM ALUMINATES (Bulletin K4)—Industrial uses of sodium aluminate are described in this two-color, 12-page brochure. Describes sodium aluminate's physical and chemical characteristics and applicability in such industries as chemical processing, cement and mortar, glass and ceramic, and rubber and textile.—Advertising Dept., Nalco Chemical Co., 6216 West 66th Place, Chicago 30, Ill.

SUGGESTED FORM OF CONTRACT FOR USE IN CONNECTION WITH ENGINEERING CONSTRUCTION PROJECTS—1960 edition prepared jointly by the Associated General Contractors of America and by the

Design of Concrete Pavements

Recommended Practice for Design of Concrete Pavements (ACI 325-58) covers the design of rigid airport and highway pavements and bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Includes recommendations for soil foundations, selection of slab dimensions, joints, and details for reinforced or nonreinforced concrete. 36 pp. $1.00 per copy, 50¢ to ACI members.

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American Society of Civil Engineers. Useful to consulting engineers and to state, county, municipal, and other awarding agency personnel who are not now using contract forms employing the most up-to-date and currently accepted practices and provisions. Costs 25 cents per copy.—The Associated General Contractors of America, Inc., 1957 E. St., N.W., Washington, D.C.

THIOKOL APPLICATION HANDBOOK — 32-page handbook of application methods to guide the user of Thiokol LP-Epoxy concrete adhesive in the proper surface preparation, mixing procedure, and equipment required. Covers the repair and maintenance of highways, buildings, bridges, airport runways, dams, sidewalks and driveways.—Dr. R. E. Robinson, Laboratory, International Epoxy Corp., 501 N.E. 33rd St., Fort Lauderdale, Fla.

REX STEEL FORMS (Catalog No. 60-163)— Detailed information on the complete line of Rex curb, combination curb and gutter, sidewalk, and flexible steel forms is presented in this 20-page catalog. Cites availability of fast, custom-made division plate service to meet special requirements.—Chain Belt Co., Milwaukee, Wis.

BUCKET ELEVATORS FOR CEMENT MILLS (Book 2714)—Publication is devoted to the Link-Belt Type 14 centrifugal discharge bucket elevators and their use in cement mills and similar applications. The 12-page book contains comprehensive engineering data on such components as SS bushed chains, chain attachments, Style AC bucket sizes, segmental rim sprockets and traction wheels and drives.—Link-Belt Co., Prudential Plaza, Chicago, Ill.

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PERLITE CONCRETE BACKUP FOR CURTAIN WALLS—The Perlite Design News, V. 60, No. 5, 1960, describes the characteristics of perlite concrete backup walls. Includes diagrams of typical spandrel sections incorporating the use of perlite insulating concrete in curtain wall construction.—Perlite Institute, Inc., 45 West 45th St., New York, N.Y.

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